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Evaluation of Precast Panels for Airfield Pavement Repair

Phase I: System Optimization and Test Section Construction

Peter G. Bly, Lucy P. Priddy, Christopher J. Jackson, and Quint S. Mason

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Abstract

During the period of March through November 2011, researchers of the U.S. Army Engineer Research and Development Center (ERDC) and the U.S. Air Force Research Laboratory (AFRL) reviewed the use of precast concrete panels for pavement repair applications in the U.S. and around the world. Based on this review, an USAF designed prototype system was selected for field investigation and was modified to allow for more efficient panel construction and installation for emergency and contingency portland cement concrete (PCC) pavement repairs. Seven precast concrete panels were fabricated for use in an airfield designed PCC test section. Three different prospective repair configurations were evaluated in a simulated airfield using the test panels. Work task items were timed during panel installation to aid in the evaluation of the repair technique effectiveness and to identify areas for optimization. Additional refinements to the system components and installation procedures were made following the field study.

Results of this phase of the investigation were used to develop fabrication and installation procedures and to determine the supplies and equipment required to construct, stockpile, and install panels in the field. A listing of the expendable construction materials and equipment items required to assemble a deployable containerized kit capable of furnishing a minimum of 12 precast panels was generated.

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Preface

Personnel of the U.S. Army Engineer Research and Development Center (ERDC), Geotechnical and Structures Laboratory (GSL), Vicksburg, MS, were tasked to evaluate the use of precast portland cement concrete (PCC) panels for rapidly repairing damaged PCC airfield pavements. Users of the information presented within this report include the U.S. Air Force's (USAF) pavement evaluation teams, contingency readiness groups, base civil engineers, major command pavement engineers, Rapid Engineer Deployable, Heavy Operational Repair Squadron, Engineer (RED HORSE) Squadrons, and Prime Base Engineer Emergency Force (BEEF) Units. Additional users of this report include Army, Navy, and Marine Corps units charged with the repair and sustainment of airfield pavements.

The project described in this report was funded by the Air Force Civil Engineer Center (AFCEC). The technical manager for this project was Dr. Craig Rutland of the AFCEC, Tyndall Air Force Base in Florida.

The findings and recommendations presented in this report are based upon field experiments conducted at the ERDC during March through November 2011. The principal investigators for this project were Peter G. Bly and Lucy P. Priddy of the Airfields and Pavements Branch (APB); GSL and Christopher J. Jackson of Applied Research Associates (ARA). The lead engineering technician was Quint S. Mason of the APB. Instrumentation support was provided by Tony N. Brogdon and Harold T. Carr from the Information Technology Laboratory at the ERDC. Technical assistance for this work was also provided by various civil engineering technicians and summer students of the APB, personnel from the Department of Public Works, and the Material Testing Center at the ERDC. Additional field testing support was provided by personnel of the Air Force Research Laboratory during panel construction and installation. Bly, Priddy, Jackson, and Mason prepared this report under the supervision of Dr. Gary L. Anderton, Chief, APB; Dr. Larry N. Lynch, Chief, Engineering Systems and Materials Division, GSL; Dr. William P. Grogan, Deputy Director, GSL; and Dr. David W. Pittman, Director, GSL.

COL Kevin J. Wilson was the Commander and Executive Director of the ERDC. Dr. Jeffery P. Holland was the Director.

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Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
gallons (U.S. liquid)	3.785412 E-03	cubic meters
horsepower (550 foot-pounds force per second)	745.6999	watts
inches	0.0254	meters
foot-pounds (force)	0.7375621493	newton meters
pounds (force)	4.448222	newtons
mils	0.0254	millimeters
ounces (U.S. fluid)	2.957353 E-05	cubic meters
pounds (mass)	0.45359237	kilograms
pounds (force) per foot	14.59390	newtons per meter
pounds (force) per inch	175.1268	newtons per meter
pounds (force) per square inch	6.894757	kilopascals
square feet	0.09290304	square meters
tons (force)	8,896.443	newtons

1 Introduction

Problem

In contingency environments, flight operations must be restored in the shortest timeframes possible, often with only 4 to 6 hrs available to complete repairs. Traditional portland cement concrete (PCC) airfield repair methods using conventional PCC, high early-strength concrete, or proprietary cementitious rapid-setting repair materials are the most broadly practiced methods. The work required to complete PCC cast-in-place repairs is fairly universal and requires minimal equipment; however, reopening times are longer because of conventional PCC's slower strength gain and difficulty to be placed in all weather conditions (Ashtiani et al. 2011). Military engineers need expedient methods for conducting partial- and full-slab replacements of damaged PCC pavements in contingency environments.

Proprietary rapid-setting repair materials have been used successfully for partial- and full-slab replacements within the objective repair timeframe; however, these materials are expensive and are a logistical burden to transport to remote locations. Repair teams may have limited quantities of these materials on site, limiting their use to small patches (less than 5 ft^2 of surface area) or for large volume repairs required to provide a minimum operating strip. Additionally, high early-strength concrete may not be as durable as traditional PCC and may not be available in a contingency environment and also may be difficult to place in all weather conditions.

Precast concrete panel technology offers a repair method to potentially eliminate the issues of traditional cast-in-place PCC repair in contingency environments. Panels can be precast in anticipation of repair activities using locally available, conventional PCC that is allowed to cure to its ultimate strength, and stockpiled on site for future use. Panels can be designed to be easily placed with readily available construction equipment within a narrow construction/repair window. Panels can be placed alone or in series with additional precast panels. This technology must be tested to see if it can meet the challenges of expedient pavement repair and withstand simulated aircraft traffic.

Prior to recommending the use of precast panels for PCC pavement repair, a literature review was conducted to identify a current repair system or to aid in developing a new system that could be used in the envisioned environment. Once a system was selected, field testing was completed to validate the performance of the panels under simulated aircraft traffic. Based on the test results, guidance for preparing and using precast panels for airfield pavement repairs was provided to military personnel.

Objective and scope of the current investigation

The objective of this research was to develop a precast PCC system for expedient airfield repairs in contingency environments. This report documents the first phase of this project, including the precast system selection, component optimization, test section construction, and field installation of the panels.

Attributes for an acceptable system (precast panels or additional supplies) include:

- Can be easily fabricated under adverse field conditions,
- Uses locally available items that are required in large quantities or represent significant costs such as aggregates, cement, etc.,
- Is suitable for long-term storage,
- Is ready for use at a moment's notice,
- Can be rapidly installed in multiple repair geometries, and
- Provides sufficient service life under the prospective aircraft loadings including C-17 and F-15E aircraft.

To identify candidate systems, information was gathered on precast PCC panel systems that have been previously used in road and airfield construction or repair. This effort led to the selection of a single prototype system for further consideration. A demonstration of this system was conducted at the U.S. Army Engineer Research and Development Center (ERDC) in July 2011. This allowed the researchers to view the precast panel prototype design, materials, and the construction tasks required to assemble a collection of precast PCC panels. Challenges and issues encountered in the use of the prototype systems were identified, and components of the system were modified to mitigate these challenges/issues and to facilitate construction and use of the system.

The optimized system was used to construct field prepared precast panels during July-August 2011. Three different repair configurations were tested to determine the best installation procedure during November 2011. Step-by-step guidance for the construction and installation of precast panels in damaged PCC pavements was developed, and a listing of equipment and disposable supplies required to install twelve panels was prepared. The second phase of this project will focus on field performance of the different configurations of installed precast panels under simulated aircraft traffic.

2 Literature review

Deployed military personnel require expedient methods for conducting full-depth repairs in damaged PCC airfield pavements. In contingency environments and other emergency situations, damaged areas must be replaced quickly to restore flight operations in the shortest time frames possible. The repair window for these repairs is often as short as 4 to 6 hrs. Damage requiring a full-depth repair can result from either traditional pavement distresses from repeated traffic or overloading from construction errors, from environmental conditions, or from explosive blasts. Traditional pavement damage or distresses include blowups, shattered slabs, corner breaks, durability cracking, deep spalling (past mid-slab depth), deteriorating patches, and utility cuts that have the potential to damage aircraft (rated as medium- and high-severity upon inspection) (UFC 2001). Damage from explosive blasts includes deep spalls, craters, or camouflets. Regardless of the cause of damage or the repair environment, these distresses are normally repaired by removal and replacement of the damaged PCC and sublayers.

The most broadly used materials for repairing PCC pavements are conventional PCC, high early-strength concrete, and proprietary rapid-setting repair materials (Williams et al. 2011; 2012). Current military guidance for conducting full-slab replacement and full-depth repairs in PCC airfield pavements suggests using conventional PCC (UFC 2001). Generally, conventional PCC provides the best results when conducting permanent repairs in PCC because the replaced material has similar mechanical properties to that of the parent pavement.

Disadvantages to using conventional cast-in-place PCC include long curing durations required to gain strength and the inability to place in all weather conditions (Ashtiani et al. 2011; Priddy and Rushing 2012). Additionally, over the past several years, the performance of proprietary rapid-setting rigid repair materials has improved, making their use acceptable for a wide range of repair types including emergency, temporary, and permanent airfield repairs (Hammons and Saeed 2010; Priddy 2011).

Proprietary rapid-setting repair materials have been successfully used for partial- and full-slab replacements for contingency repairs; however, these materials are expensive and are a logistical burden to transport to remote locations (Priddy and Jersey 2009). Military repair teams in a contingency environment may have limited quantities of these materials; thus, alternative technologies that take advantage of local materials are desired.

High early-strength concrete has gained acceptance in the commercial and military airfield repair communities in recent years. The combination of high cement content and the use of accelerating admixtures yields repairs that can be reopened to traffic within 6 to 12 hrs. High early-strength concrete typically costs more than traditional PCC but is usually less expensive than proprietary rapid-setting repair materials. In addition to cost, durability is a concern, not only from the high cement contents required, but also from the questionable quality of materials often encountered in contingency environments (Williams et al. 2011). Furthermore, high early-strength concrete and proprietary repair materials have the same drawback as traditional PCC, as they cannot be placed in all weather conditions (Ashtiani et al. 2011).

A promising alternative repair method is the use of precast PCC panels for full-depth repairs. Precast PCC panels may provide a higher quality repair than that achieved using conventional, proprietary, or high early-strength concretes since the panels can be prepared with locally available, conventional PCC and stockpiled for later use. More time would be available to fabricate each panel in advance of an emergency repair scenario, so the panel can be prepared in less haste than current emergency repair methods. Additionally, conventional PCC may be more economical than using the expensive proprietary materials in terms of material costs (Hossain et al. 2006).

Precast PCC panel usage

Modular or precast structural elements such as concrete columns, beams, piles, highway barriers, and railroad ties are used extensively in the building, highway, and bridge industries (FHWA 2007; Rollings and Chou 1981). Precasting and storing these elements away from the construction site can reduce congestion at the job site, and their mass production in a factory-like setting can result in improved quality control and minimized costs (Rollings and Chou 1981). The use of precast concrete slabs or panels in conventional road and airfield pavements for either pavement construction or pavement repair is not a recent innovation, and various studies have been conducted over the last 80 years. One reason usage of these panels has lagged behind other precast structural elements is that the precast panels used for pavements require more effort to place foundation or bedding materials in such a way as to maintain full and uniform underlying support for the panel (Kohler et al. 2009; Rollings and Chou 1981). Furthermore, the bedding material must be placed in such a manner to ensure panels match the elevation of the surrounding pavement. Careful base preparation is not only time consuming but also requires experienced field crews and heavy equipment. As a result, this repair method was not necessarily considered practical for contingency or emergency repair efforts.

Another reason reported in the literature stating that precast pavement repairs have lagged behind other repair techniques is the lack of documented design and construction practices for precast panels (Tayabji et al. 2011). While proprietary and non-proprietary panels exist and have been explored in recent years, there has been hesitancy by pavement engineers to use them on an extensive basis because of the lack of adequate documentation for their successful construction and long-term performance. Finally, the use of panels may be more expensive compared to other repair or construction techniques because of the need for heavy construction equipment, use of proprietary panel systems and equipment, and specially trained crews.

Early precast panel experiences

A review of the literature reveals usage of precast PCC slabs (or panels) for a variety of single- and multiple-panel repairs as well as rapid pavement construction in North America, Europe, the former Soviet Union, and Asia, as early as the 1930s. Precast panels were used for airfield construction in the former Soviet Union as early as the 1930s and in Europe from 1947 through 1958 (Rollings and Chou 1981). Since this time, the majority of research in the U.S. using precast panel technologies has been for highway repairs. Summaries of early precast panel use are provided by Rollings and Chou (1981), Brabston (1984), Federal Highway Administration (FHWA 2007), Kohler (2007), and Tayabji et al. (2009; 2011). Table 1 summarizes the early international uses of precast panels; Table 2 presents a summary of the U.S. precast panel efforts through the 1980s. As can be seen in these tables, panel designs, seating methods, load transfer mechanisms, and reinforcement types varied greatly among the reported efforts.

Table 1. Early international highway and airfield efforts using precast panels.

Time Period	Location	Pavement Type	Dimensions	Comments	References
1930s-1980s	Soviet Union (multiple locations)	Airfield construction and road construction	13-20 ft x 6 ft (airfield)	Multiple panel sizes used for both airfield and highway construction	Rollings and Chou (1981) Brabston (1984)
1947	Orly Airport, Paris, France	Airfield construction	3.3 ft x 3.3 ft x 6.3 in.		Rollings and Chou (1981) Brabston (1984)
1956	London, England	Airfield construction		Dimensions not reported	Brabston (1984)
1956	Finningley, England	Airfield Construction	30 ft x 9 ft x 6 in.	Prestressed, precast slabs	Brabston (1984)
1958	Melsbroek, Belgium	Airfield	39 ft x 4.1 ft x 3 in.	Prestressed, precast slabs	Brabston (1984)
1980s	Germany	Airfield emergency repair	6.56 ft x 6.56 ft x 4.72-5.91 in.	Simulated munition blast repairs	Brabston (1984)
1970s	Japan	Airfields and container yards		Sizes not reported.	Kohler et al. (2007)
1981	Japan	Airfield	3.2 ft x 7.5 ft x 7.9 in.	Designed for DC-8 traffic.	Brabston (1984)
1991	Japan	Road	3.3-9.8 ft x 6.6 ft x 5.9 in.	No load transfer devices reported.	Brabston (1984) Kohler et al. (2007)

Table 2. Early U.S. highway and airfield repair efforts using precast panels.

Time Period	Location	Pavement Type	Dimensions	Comments	References
1960s	South Dakota, United States	Highway	24 ft x 6 ft x 4.5 in.	Panels were overlaid with 1.5-3.5 in. of AC. Panels were prestressed.	Rollings and Chou (1981)
1970s	Michigan, United States	Highway	10-11 ft x 6-12 ft x 8-9 in.	Doweled and undoweled panels	Rollings and Chou (1981) Simonsen (1971, 1972)
	New York, United States	Highway	20-30 ft x 12-13 ft x 9 in.	Pretensioned precast panels	Overacker (1974)
	Florida, United States	Highway	20 ft x 12 ft x 8 in.	Panels raised into place using slab jacking. Used to conduct interstate repairs.	Grimsley and Morris (1975)
	California, United States	Freeway	12.3-17.4 ft x 11.4 ft x 8 in.	Grout bedding and grout filled joints	Better Roads (1974)

Time Period	Location	Pavement Type	Dimensions	Comments	References
	Virginia, United States	Highway	1-3 ft x 1-2 ft x 2 in.	Conducted 68 partial depth patches using precast panels seated on epoxy grout	Creech (1975)
	South Dakota, United States	Highway	Unknown	Partial depth precast panels	Rollings and Chou (1981)
	New York, United States	Airfield	30 ft x 12 ft x 9 in.		Overacker (1974)
1980s	Wisconsin, United States	Highway	6 ft x 6 ft x 8.5 in.	Panels were placed on 0.5 in. of mortar grout.	Sharma (1990)
	California, United States	Airfield		116 panels were replaced, and each panel was custom built to the slab requiring replacement. The panels were overlaid with 20.3 cm of AC.	Rollings and Chou (1981) Brabston (1984)
	Florida, United States	Airfield	6 ft x 6 ft x 8-12 in.	Placed on grade and bonded with polymer concrete and or covered with polymer concrete	Brabston (1984)
	Mississippi, United States	Airfield	20-50 ft x 20-50 ft x 6-8 in.	Predicted repair times for continuous repairs to repair bomb craters.	Brabston (1984)

Recent U.S. precast panel experiences

As discussed in the previous section, infrequent precast panel investigations were conducted in the U.S. from 1970-2000; however, there has been a resurgence of investigations of this technology in the past 10 years (Tayabji et al. 2009; FHWA 2007). Until recently, the precast PCC panel systems were periodically studied for technical feasibility or as a “matter of technical curiosity” (Tayabji et al. 2009). Today, the major precast PCC panel focus is for highway or tollway repairs. Tayabji et al. (2009) provides a comprehensive summary of precast panel usage since 1995, which resulted in a resurgence of interest and investigations into precast panels for highway repairs. Major projects reported included those led by the Federal Highway Administration (FHWA), Michigan Department of

Transportation (DOT), American Association of State Highway and Transportation Officials (AASHTO), Strategic Highway Research Program (SHRP2), and commercial efforts.

FWHA/Michigan DOT

In the late 1990s, the FHWA sponsored the Concrete Pavement Technology Program (CPTP) and provided the Michigan DOT with funding to investigate the use of precast PCC panel systems for full-depth PCC repairs. This work led to the “Michigan Method” of precast slab repair discussed later in this chapter. This method is one of the most common precast PCC panel repair methods used in the U.S. for highway repairs (Tayabji et al. 2009).

Buch et al. (2003) detailed the installation of 21 precast panels for full-depth patching efforts along two interstates in Michigan. The Michigan Method panels were fabricated by a vendor following specifications for slab thickness, dowel bar, and reinforcement placement. The panel dimensions were typically 12 ft x 12 ft x 10 in. with three 1.5-in.-diameter dowel bars cast in the wheel paths to ensure load transfer across the joints.

This method uses precast panels seated on a layer of flowable fill. Dowel bars cast into the panels are connected to the surrounding pavement through dowel receptacles that are saw cut and excavated using a jack hammer. After the dowel slots (or receptacles) are prepared, the panels are lowered into place, leveled, and then the dowel receptacles are filled with high-early strength concrete or a rapid-setting proprietary material. Figure 1 presents a cross-sectional view of the dowel placement. An alternative for leveling the panels is to use high-density polyurethane (HDP) foam instead of a grout. The foam is injected through either pre-formed or drilled holes in the panels. The final step in the process is to seal the joints. For the repair, the time-consuming repair activities included preparation of the dowel receptacles, removal of the existing PCC, and the adjustment of the panel elevation with respect to the surrounding slabs.

State Highway

Numerous highway agencies have investigated the use of precast panels for pavement repair since 2000 including: Illinois Tollway Authority, New Jersey Turnpike, and New York State Thruway Authority. State DOTs

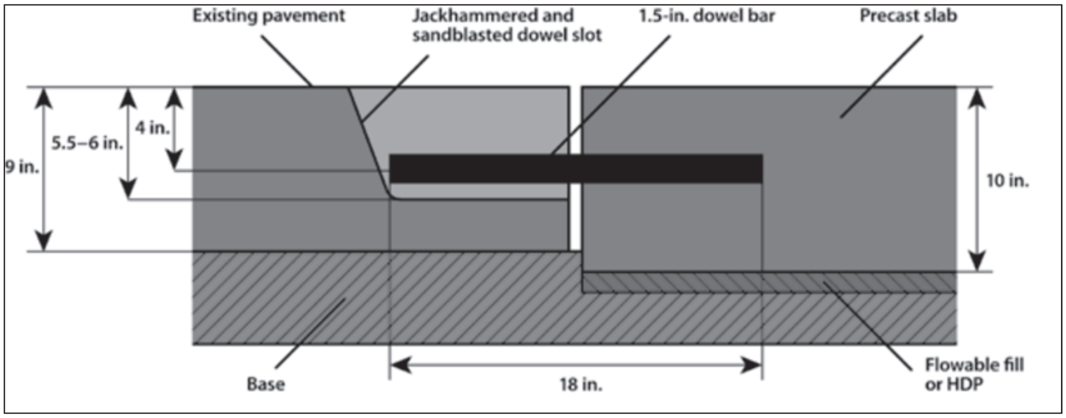

Figure 1. Michigan method cross-section of dowel assembly (Buch 2003).

included California, Iowa, New Jersey, New York, Colorado, Delaware, Florida, Hawaii, Indiana, Michigan, Minnesota, Missouri, Texas, and Virginia (Tayabji et al. 2009; Davis 2006). In 2006, the AASHTO promoted technology transfer activities related to precast PCC systems for a variety of highway paving and repair activities. In 2008, various documents were prepared and distributed regarding the use of this technology for roads. The documents provided specifications for precast PCC pavement panel systems approval, guidance, and considerations for designing precast PCC panel systems for pavements and generic specifications for fabricating and constructing the precast PCC pavement panels (Tayabji et al. 2009).

Recent airfield applications

International airports

While precast panels have been explored extensively for highway and tollway repairs in recent years, fewer research investigations have been focused exclusively on airfield pavement repairs. Some work was conducted between 2000 and 2003 at La Guardia International Airport in New York, St. Louis International Airport in Missouri, and Dulles International Airport in Washington, D.C. (Tayabji et al. 2011). Little information is available about the quality of these repairs or performance under traffic. The repairs conducted at La Guardia were conducted in test sections to simulate primary taxiway repairs. The study investigated two types of precast panels including 16-in.-thick reinforced panels and 12-in.-thick, two-way prestressed precast panels. The panel dimensions were both 12.5 ft x 25 ft and were used to construct two 200-ft-square test sections in 2002. These sections are continuing to be monitored under live aircraft loadings, and according to Oldis et al. (2009), these sections are performing satisfactorily.

Information about the performance of the precast panels at the St. Louis and Dulles airports is not readily available in the literature.

"Soviet-style" slabs

The U.S. encountered precast panel airfield pavements in countries formerly part of the Soviet Union during recent military operations in Afghanistan and Iraq (Figure 2). Research was conducted in the U.S. by Tingle et al. (2007) to understand the load-carrying capacity of the panels and to predict damage caused by U.S. aircraft. A precast panel runway using "Soviet-style" designed slabs was constructed in 2007, and a panel being placed over a sand bedding material is presented in Figure 3. Sapozhnikov and Rollings (2007) conducted a complementary study to the Tingle et al. (2007) study to understand the Soviet precast panel manufacturing and design process to determine if the technology could be applied for U.S. efforts. Only preliminary information was available on the Tingle et al. (2007) investigation at the time this report was written.

Air Force Method

Recently, the U.S. Air Force developed a prototype repair method using single precast panels referred to as the "Air Force Method" in the literature. The panels are designed such that deployed personnel can assemble prefabricated forms in the field and cast/stockpile panels on-site for future use. The precast panel dimensions are 9 ft 10.5 in. x 9 ft 10.5 in. x 11 in. Load transfer is provided by ten, 1.0-in.-diameter, 22-in.-long dowels precast into the slabs on both sides of the panel in the direction of traffic. Similar to the Michigan Method, dowel receptacles are saw cut and prepared in the surrounding PCC. Following the placement of each panel, the dowel receptacles are filled with rapid-setting cementitious repair material. Figure 4 shows the installation of the precast panels and dowel receptacles.

In a study conducted by the Air Force described by Ashtiani et al. (2010), two single-panel repairs were conducted using foam injection for leveling, and a third single-panel was seated on a layer of flowable fill backfill. Each repair was trafficked using an F-15 load cart for 1,508 passes. Load transfer was calculated for each repair before, during, and after trafficking, using deflection data obtained from heavy weight deflectometer (HWD) joint tests. The foam injected repairs provided better load transfer, but the flowable fill backfill provided sufficient support for the design traffic and was deemed suitable for contingency repairs (Ashtiani et al. 2010).

Figure 2. (left) Stockpiled precast panels at a Soviet airfield and (right) precast concrete panel runway still in service (Tingle et al. 2007).

Figure 3. Placement of a precast panel (Tingle et al. 2007).

Figure 4. Air Force method installation (Ashtiani et al. 2010).

Commercial precast panel systems in the U.S.

Currently, the most common methods of precast pavement in the United States are the Fort Miller Super-Slab Method, the Michigan Method, the Uretek Method, or some variation of these three methods. Additional commercial systems have been developed recently including the Kwik Slab system and the Roman Road System® by the Roman Stone Construction Company. Table 3 presents the main characteristics of each repair method. The following sections summarize each method.

Table 3. Main characteristics of precast panel repair methods after Ashtiani et al. (2010).

Name of repair method	Application type	Load transfer	Base support
Fort Miller Super-Slab®	Single- or multiple-panel repairs	Dowels inserted into the existing pavement	Manufactured sand and injected grout
Michigan	Single-panel repairs	Dowels cast into the precast panel and grouted in the existing pavement	Flowable fill or polyurethane foam
Uretek	Single- or multiple-panel repairs	Fiberglass ties inserted after the precast panel is placed	Grouting using injected polyurethane foam
Kwik Slab	Multiple-panel repairs/construction	Kwik Joint steel couplers	Pumped grout through grout holes and channels
Roman Road System®	Single- or multiple-panel repairs/construction	Dowels either cast into the precast panel or dowels inserted after precast pavement is placed	Grouting using injected polyurethane foam
Other	Single- or multiple-panel repairs	Dowels inserted after the precast panel is placed; other means	Any of the above

Fort Miller Super-Slab method

The Fort Miller Super-Slab® method is one of the most common proprietary precast panel methods used in the United States and Canada. The method consists of placing fabricated slabs on a carefully graded bedding material. Each panel is fabricated to exactly fit the area to be replaced. Panels are tied together through dowel receptacles that are formed in the bottom of the slabs to tie preinstalled dowels from the adjacent slabs (parent slabs). The dowel receptacles, also called slots or mouse holes, are grouted after placement with rapid-setting cementitious grout to embed the dowel bars through manufactured grout holes. The panels have additional grout holes that allow a bedding grout to be pumped under the panel through formed channels on the underside of the slab to fill any voids present and to level the panels (Fort Miller 2003). If precast panels are connected to one another, then dowel bars are cast on one side of the precast panel with corresponding dowel receptacles formed on the opposite side as shown in Figure 5. Figure 6 presents the dowel receptacles in the grouted and ungrouted conditions.

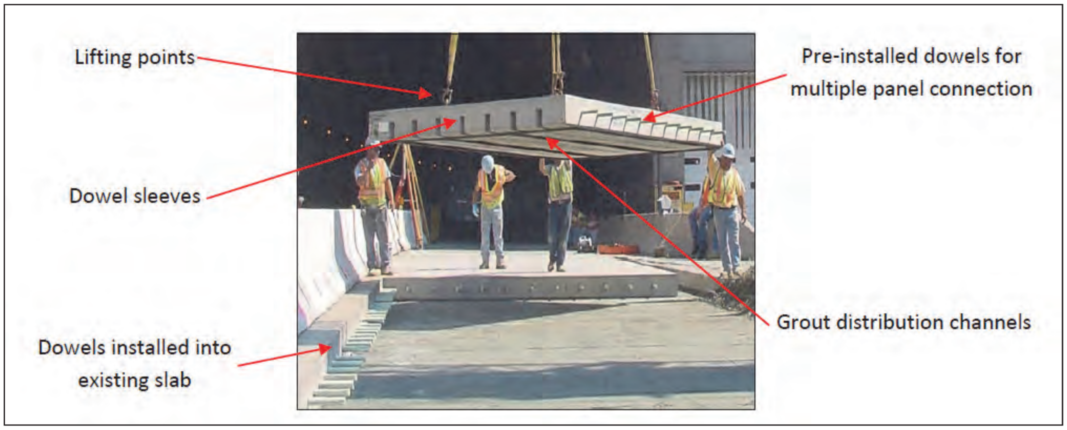

Figure 5. Super-Slab® installation at a tollbooth (Ashtiani et al. 2010).

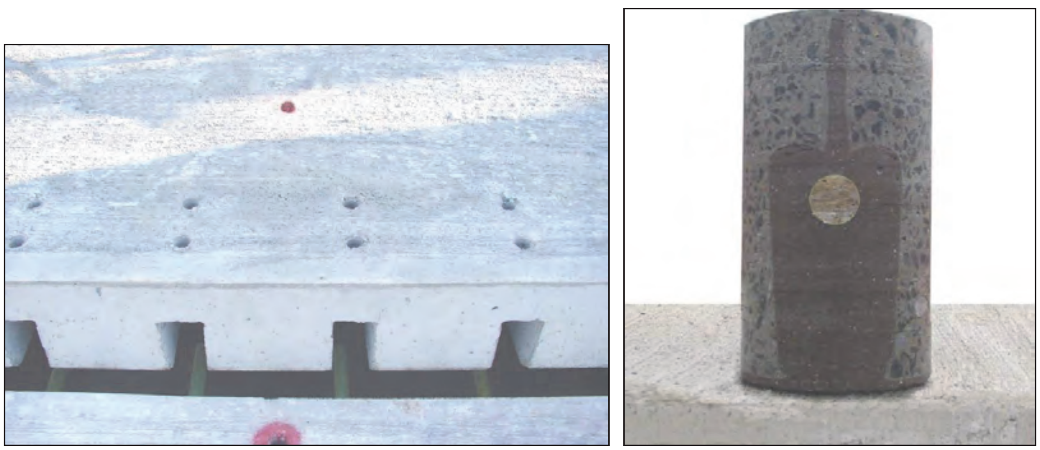

Figure 6. (left) Dowel receptacles and grout holes; (right) grouted receptacle with dowel (Thomas 2008).

Since 2002, the Fort Miller Super-Slab® method has been used by several agencies as summarized in Table 3 after Thomas (2008) and a recent SHRP2 investigation (Tayabji et al. 2011). Kohler (2008) details a precast panel test section using the Super-Slab® system trafficked using a heavy vehicle simulator in San Bernardino County, CA. The test indicated that the Super-Slab system could support an estimated 140 to 240 million equivalent single-axle loads (ESALs), equivalent to more than 25 years of highway service. Failure was considered similar to jointed plain PCC with corner breaks resulting from loss of support. Tayabji et al. (2009) reports that repairs conducted using this method have lasted over 7 years in highway applications with no issues.

Table 4. Super-Slab® installations after Thomas (2008) and Tayabji et al. (2011).

Project	Owner	Area (ft²)	Work Window	Nature of Repair	Date of Installation
Tappan Zee Bridge Toll Plaza	New York State Thruway	158,000	Off-peak hr	Multiple- panel	2002
Dulles Airport Taxiway	Metropolitan Washington Airport Authority	3,500	8 hr (night)	Single-panel	2002
9A Ramp, Tarrytown, New York	New York State Thruway	15,750	Day and night	Multiple-panel	2003
Lincoln Tunnel, New Jersey	Port Authority New Jersey	8,100	Weekend	Single-panel	2003
Belt Parkway Ramps, Jamaica, New York	New York State DOT	16,030	Full Closure (1 month)	Multiple-panel	2003
Korean Veterans Parkway, Staten Island, New York	New York State DOT	8,850	8 hr (day)	Single-panel	2003
Toronto, Ontario	Ministry of Transportation, Ontario, Canada	1,220	8 hr (night)	Single-panel	2004
Port Jefferson, New York	New York State DOT	2,650	8 hr (night)	Cross walks	2005
I-90 - Albany, NY	New York State DOT	56,400	8 hr (night)	Single-panel	2005
Fontana, California	CALTRANS	1,950	Off highway	Test section	2005
Minneapolis, Minnesota	Minnesota DOT	2,592	Full closure	Multiple-panel	2005
Marine Parkway	Metropolitan Transit Authority (New York)	2,592	3 day full closure	Multiple-panel	2005
Fordham Road, Bronx, New York	New York State DOT	3,852	8 hr (night)	Multiple-panel	2006
Route 7 Cross Town, Schenectady, New York	New York State DOT	26,586	10 hr (night)	Intersection	2006
High Speed EZ Pass Slabs	New York State Thruway	576	8 hr (night)	Special	2006
Schuylerville, New York	New York State DOT	1,152	Off highway	Trial	2006
Southern State Parkway	New York State DOT	2,483	8 hr (day)	Single-panel	2007
I - 95, New Rochelle, NY	New York State Thruway	40,000	5 hr (night)	Single-panel	2007
Chicago, Illinois	Illinois Tollway	768	Off highway	Trial	2007
I-295, Trenton, New Jersey	New Jersey DOT	2,300	8 hr (night)	Single-panel	2007
I-88, Chicago, Illinois	Illinois Tollway	476	Not specified	Single-panel	2007
I-294, Chicago, Illinois	Illinois Tollway	2,674	Not specified	Multiple panel	2007
1-88, Chicago, Illinois	Illinois Tollway	4.338	Not specified	Multiple-panel	2008
I-295, Burlington County, New Jersey	New Jersey DOT	30,395	Not specified	Single-panel	2007/2008
Route 21, Newark, New Jersey	New Jersey DOT	69,810	Not specified	Single- and multiple-panels	2008
I-280, Essex County, New Jersey	New Jersey DOT	38,000	Not specified	Single-panel	2009
Route 42, Camden and Gloucester, New Jersey	New Jersey DOT	32,034	Not specified	Single- and multiple-panels	2009
Nassau and Suffolk Counties, New York	New York DOT	3,640	Not specified	Single-panel	2009

Project	Owner	Area (ft^2)	Work Window	Nature of Repair	Date of Installation
Memorial HWY and Division Street, New Rochelle, New York	New York DOT	3,041	Not specified	Multiple-panel repairs	2008
Nassau Expressway, Queens, New York	New York DOT	85,000	Not specified	Multiple-panel	2009
I-15, Utah	Utah DOT	28,800	Not specified	Single- and multiple-panels	2009
US 60 I-66 Ramp, Fairfax, VA	Virginia DOT	432	Not specified	Single- and multiple-panels	2009
I-15, Ontario, California	CALTRANS		Not specified	Multiple-panel	2010

Uretek Method

Uretek USA, Inc. developed a repair method for leveling in-place slabs in the late 1990s. The method requires injecting HDP foam through holes drilled through the PCC surface into the sublayers to lift faulted slabs to match the surrounding pavement's elevation. This process is known as the Uretek Method. This method has also been applied to contingency airfield repairs using cast-in-place, rapid-setting material as a repair procedure developed by the U.S. Navy (Priddy et al. 2010). Recently, the injection leveling method has also been applied for precast panels for both the previously described Michigan and Air Force Methods.

For precast panels, the foam injection method may be combined with the Stitch-In-Time® Process also developed by Uretek. The Stitch-In-Time® Process is used to restore load transfer in jointed concrete pavements. For precast panel repairs, a precast panel is lowered into the area where damaged pavement has been removed with little or no bedding preparation. The slab is then leveled by injecting the foam under the slab through either formed injection ports placed during panel construction or drilled holes after slab installation. Once the slabs are leveled, the Stitch-In-Time® Process is used to provide load transfer between the precast panel and the surrounding pavement (Tayabji et al. 2009; Ashtiani et al. 2010). The panels are "stitched" to the existing slab or to another panel using fiberglass ties that serve as load transfer mechanisms. The ties are inserted and grouted into receptacles that extend from the existing slab to the precast panel or precast panel to precast panel. Figure 7 shows the Uretek fiberglass tie installation procedure.

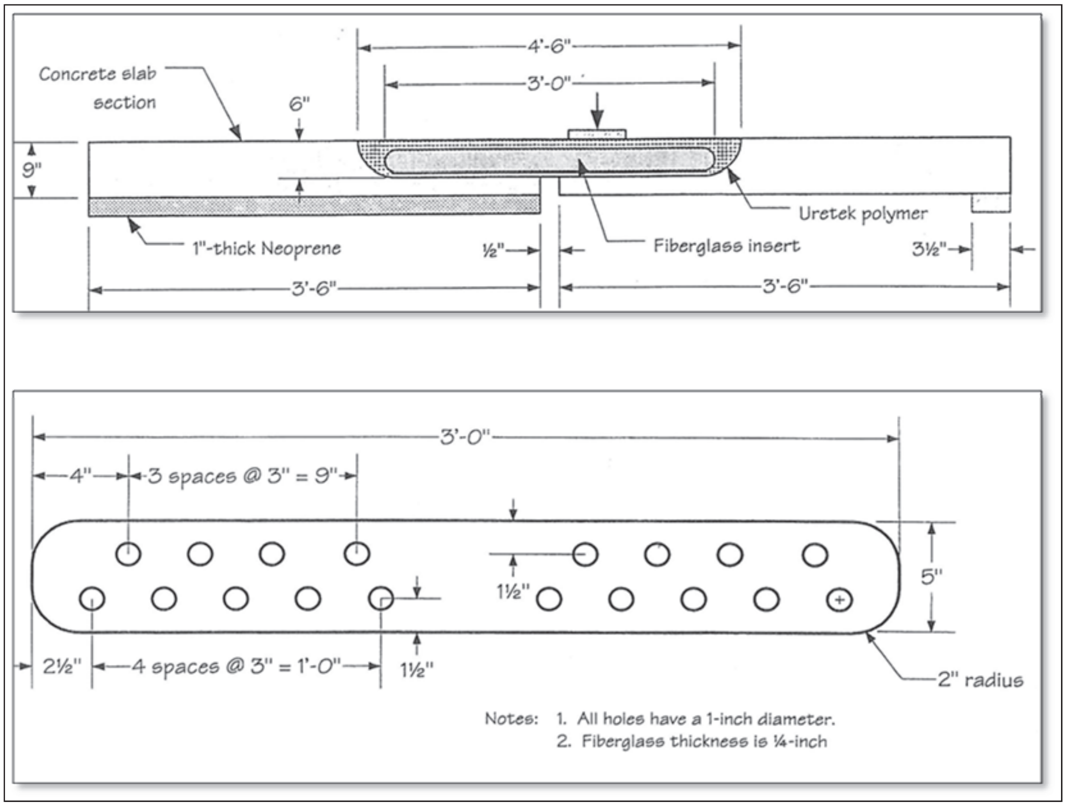

Figure 7. Uretek fiberglass ties after Ashtiani et al. (2010).

The U.S. Air Force evaluated the Uretek Method without the fiberglass ties for leveling precast panels in 2009. The Air Force prototype panels used dowels for load transfer in lieu of the fiberglass ties. However, the researchers did not recommend this procedure for expedient airfield repair applications because of the precision and training that would be required to properly use the foam injection system (Ashtiani et al. 2010).

Kwik Slab System

The Kwik Slab System is a newer precast panel repair system developed and marketed in Hawaii and Singapore. The system relies on the placement of precast panels that are connected together using Kwik Joint steel couplers that are precast in one or more panel ends. The panels are leveled using the Uretek HDF injection, through plastic leveling shims, or grout leveling pads (Kwik Slab 2011). The couplers are then grouted with a high-strength grout. The joint details are presented in Figure 8. At the time of this report, little information was available to the performance of this system.

Figure 8. (left) precast panel and (right) Kwik Joint connection (Kwik Slab 2011).

Roman Road System®

The Roman Road System® was developed by the Roman Stone Construction Company in 2009. Panels are placed in the prepared repair area then leveled using injected HDF in the same manner as the Uretek or Michigan seating methods. The HDF used is an Uretek product (Uretek 600). Load transfer is provided by saw cutting through both the parent PCC and the precast panel, and installing load transfer devices following the leveling of the panel. The panels are typically 1 in. thinner than the existing pavement (Tayabji et al. 2011; Roman Stone Co. 2012). The installation process is presented in Figure 9. At the time of this report, little information was available to the performance of this system.

Figure 9. (Left) installing Roman Road slab; (right) injecting polyurethane foam (Roman Stone Co. 2012).

Recent world experiences

In addition to the recent U.S. experiences, a resurgence of precast panel research has occurred around the world. Kohler et al. (2007) and Tayabji et al. (2011) present summaries of recent worldwide precast panel experiences.

Netherlands

Several pilot factory-produced modular road surface studies were conducted in the Netherlands during 2004-2006 (van Dommelen et al. 2004). One precast panel concept investigated was a modular pavement structure called ModieSlab. For this method, precast concrete panels are designed to either rest on concrete piles or are laid within existing pavement. The ModieSlab dimensions are 11.6 ft x 8.3 ft x 12.6 in. and do not use load transfer devices (Smits 2004). Houben et al. (2004a, b) and van Dommelen et al. (2004) detail a pilot study and accelerated pavement testing study using ModieSlab (among other technologies). In the pilot study, the ModieSlabs were constructed on a highway in the Netherlands. Problems encountered included smoothness, raveling, and polished aggregate.

France

Kohler et al. (2007) detailed a study conducted in France using hexagonal shaped panels placed over a granular bed (Figure 10). This precast panel research was part of a Removable Urban Pavements research project coordinated by the Laboratoire Central des Ponts et Chaussees in France. The method consisted of using small hexagonal panels that are connected together to form the pavement so that when a panel is damaged, it can be removed and replaced with limited equipment and resources (Green Link 2010). Each slab was 8-in.-thick and had an equivalent diameter of 5 ft.

Indonesia

Tayabji et al. (2011) summarized a construction project in Indonesia in 2008 using precast concrete panels to construct a four-lane toll road across 22 miles of remote terrain. The panel dimensions were roughly 8.2 ft x 27 ft x 8 in. and were placed on 2 in. of a lean concrete base and required post-tensioning.

Japan

Tayabji et al. (2011) also described the use of precast PCC pavements in Japan in recent years. For Japanese road applications, precast panels were placed on an asphalt layer to prevent pumping and any gaps are filled

Figure 10. Accelerated pavement testing of hexagonal slabs in France (Kohler et al. 2007).

using grout. Standard slab dimensions used were 18 ft x 4.9 ft x 8-10 in. The Japanese precast panels relied on a load transfer device called the "horn device" (Hachiya 2001). For airport applications, the panels were 47 ft x 8 ft x 10 in, prestressed, and some incorporated both the "horn device" and a compression joint for load transfer. The Japanese also developed an innovative sliding dowel bar for airport construction using precast panels. Details of the Japanese precast panels are provided by Hachiya (2001) and Nishizawa (2008).

SHRP 2 investigations

From 2008 through 2012, the Strategic Highway Research Program 2 (SHRP2) Project R05-Modular Pavement Technology was conducted to develop guidelines and to synthesize precast pavement information. The ultimate goal of the project was to encourage the adoption of precast repair and construction techniques by the paving industry. The study focused almost exclusively on precast panel technologies to develop guidance that would allow transportation agencies to design, construct, install, maintain, and evaluate modular pavement systems.

As part of this project, a number of in-service precast panel systems were investigated in the U.S. Preliminary results reported by Tayabji et al. (2011)

indicated that as long as the panels were installed correctly (proper bedding, dowel alignment, etc.), they have the potential to provide long-term service for pavement repairs of 15 to 20 years for highway applications. Field tested precast panel repairs were reexamined as part of this study. Results of inspections emphasized the need for good support under the repair and care when installing dowel-receptacle patching materials. The study also recommended using dowel bar caps to minimize failure of the dowel bar receptacle patches.

Precast panel installation process

In general, the precast panel installation process regardless of pavement type (highway or airfield) can be summarized with the following activities or steps:

- Identify distresses requiring repair
- Establish and mark repair boundaries
- Select panel size(s)
- Fabricate panel(s)
- Saw cut panel boundaries and dowel receptacles (if required)
- Remove existing pavement and prepare dowel receptacles (if required)
- Clean dowel receptacles
- Prepare subbase
- Place leveling/bedding material
- Install panel
- Adjust panel elevation to match surrounding pavement/panels through reseating or injection of foam/grout, depending on method
- Install load transfer if not precast in panels
- Grout dowel receptacles with high-early strength PCC or proprietary material
- Seal joints with joint sealant or grout
- Check installed panel for elevation differences

Advantages and disadvantages of precast paving technology

This section summarizes the general advantages and disadvantages of using precast PCC slabs for full-depth pavement repair as gleaned from the literature reviewed.

Advantages of using precast PCC slabs

- Improved quality of precast panels compared to hastily prepared cast in-place patches
- Uses locally available materials
- Potentially less expensive than proprietary rapid-setting repair materials
- Reduced overall repair time. Little to no cure time required for cementitious materials
- Ability to place panels in most weather conditions

Disadvantages of using precast PCC slabs

- Potential for higher cost of repairs, especially for smaller projects
- Potential for slower field installation rate for multiple-panel repairs
- Requires experienced equipment operators and heavy equipment such as cranes for installation
- Precast panel size restrictions because of to lifting capabilities or transport dimensions
- Seating and leveling problems may require repeated placement and removal of precast panels and/or slab grinding
- Time and precision required for dowel receptacle cutting
- Time and precision required for panel insertion
- Warped panels are required for non-planar subsurfaces
- Production of foreign object debris (FOD) from dowel receptacle or panel grouting
- Dowel alignment concerns
- Uncertain long-term performance

In addition to the advantages and disadvantages of precast panels, Tayabyi et al. (2009) provided additional technical and institutional challenges for precast PCC pavements as presented below:

- *Lack of understanding of the load carrying capacity of each system component, seating and support conditions, load transfer at joints between multiple panels and single panels and the existing pavements, and connectivity at joints;*
- *Lack of optimization for various system design features;*
- *Ensuring durability;*
- *Lack of adequate long-term performance history;*
- *Lack of component testing, such as joint connectivity;*

- *Availability of production/assembly plants for panel fabrication;*
- *Lack of well-developed QC procedures;*
- *Lack of well-developed QA procedures, including panel dimension tolerances, profile, load transfer effectiveness at joints, and initial faulting at joints;*
- *Lack of treatment procedures for early failures;*
- *Opening to traffic requirements;*
- *Maintaining safe riding surface;*
- *Maintaining vertical alignment of joints;*
- *Lack of best practices for design, construction, maintenance, and rehabilitation for precast systems;*
- *Lack of well-developed specifications for using precast systems; and*
- *General lack of support by precast concrete industry to support refinement of precast systems.*

While these concerns were focused mainly on highway applications, there is a lack of understanding if precast panels can carry heavy aircraft loads, and if currently available systems can be modified to suit temporary or permanent repairs of airfield pavements. Little information is available in the literature regarding the long-term performance of precast panels in an airfield setting. Some areas of particular concern are the opening to traffic requirements for emergency airfield repairs, joint connectivity and load transfer for multi-panel repairs, and elevation differences that may exist between precast panels and the surrounding pavements. A final concern is the production of FOD from panels requiring grouting or patching of dowel receptacles.

Summary

There has been resurgence in the use and study of precast PCC slabs for highway and airfield repairs in recent years. The majority of research in the U.S. has been focused on highway applications of precast PCC panels. This chapter summarized worldwide experiences using precast PCC slabs for pavement repairs. Since the 1930s, various procedures have been developed and investigated around the world, and these procedures are still evolving as more experience is gained through research and trial sections. Current systems vary based on slab size, reinforcement design, bedding materials, and load transfer mechanisms. The advantages of using precast pavement technology include higher quality concrete repairs through use of precision fabrication, minimal weather restrictions, and reduced delay in reopening a pavement to traffic. The main disadvantages of precast panel repairs include lifting capabilities in the field, seating and leveling issues, and finally, the uncertainty of the long-term performance of the current systems. Continued research and trial sections of technologies will be required as methods are refined or new methods are developed to refine the current processes.

3 Selection of a precast panel system

As presented in Chapter 2, the literature reveals numerous investigations and currently available commercial systems for use of precast panels for pavement repair. As part of this research, the operational needs of a precast airfield pavement repair system were weighed against the information collected in Chapter 2 to select or design a precast system that best fits the U.S. Air Force requirements for development of a single repair method for contingency airfield repairs.

U.S. Air Force system requirements

The Air Force Civil Engineer Center (AFCEC) requested the development of a precast panel repair system for repair of rigid airfield pavements in contingency situations. Design criteria were provided by AFCEC to assist with the system design. The precast panel system criteria were as follows:

- Must support F-15E and C-17 aircraft loadings (3,700 passes),
- Must be versatile for varying repair sizes including partial- and full-slab repairs,
- Should maximize use of available equipment at a typical Air Force base with additional equipment and materials provided in a deployable containerized kit,
- Should rely upon simplified tactics, techniques, and procedures (TTPs) that require minimal technical training , and
- Allow modular formwork for preparing panels at a contingency location.

Initial system selection

To accommodate AFCEC's requests, the information within Chapter 2 was used to determine if a current system could be used or optimized for the Air Force through technology demonstrations.

For expeditionary locations, commercial systems produced in a factory setting will not meet DoD mission requirements. Factory production requirements will create a logistical burden to transport panels to a remote destination. Also, various commercial U.S. systems require special equipment to assist with leveling and filling the voids beneath the panels.

Bedding material injection systems, GPS guided (laser) screeds, and precision leveling equipment would not be easily applied without procurement and proper training of personnel conducting the work. The significant equipment demanded makes many commercial systems unusable for most contingency repair operations as described in the literature.

Some repair methods/systems presented in Chapter 2 rely on prestressing the panels. The majority require pre-tensioning individual panels during construction to minimize cracking and to allow for larger panel dimensions. Other options add additional post-tensioning after installation to tie multiple panels together. Since the goal is to develop a system for contingency repairs, systems with post-tensioning are not applicable because multiple panels may not be placed next to one another (single-panel or partial-slab repairs). Additionally, post-tensioning of installed panels will likely cause installation times to be beyond the limit the Air Force finds acceptable.

Pre-tensioning of panels during their construction is also undesirable because it adds complexity to the construction process. The system design and reinforcement plan should be basic enough that a wide cross-section of users can assemble the slabs in the field. Because the system constructed will focus on smaller, repair sized panels that must have adequate thickness to support aircraft traffic, the need for pre-tensioning is not required.

A system utilizing a HDP foam bedding layer may also be difficult to implement. The use of injected polyurethane foam has been investigated for use in multiple repair methods including the U.S. Air Force, Uretek Method, Michigan Method (injection method), Kwik Slab, and Roman Road. The injection of foam requires the use of highly trained personnel to inject the foam to prevent cracking of the panels, excessive lifting, and damage to surrounding pavements. This step in the repair process also requires additional equipment and materials that may be difficult to maintain, operate, and store in a contingency environment. Because of the sensitivity of the injection process to the outcome of the finished repair, storage concerns, and equipment/training requirements, methods that rely on injected foam leveling are not recommended.

Methods presented outside the U.S. were also eliminated from consideration because of the lack of information regarding the performance of the panels and design methods. For this reason, neither the ModieSlab nor the French hexagonal slabs were recommended for further consideration.

After applying the design criteria and eliminating the various repair methods reviewed, only the Michigan Method and Air Force method utilizing flowable fill for bedding material were further examined. The components of both methods are fairly similar and have both been used for single-panel repairs. The Michigan Method is designed for roadway use. Dowels are only located in the wheel paths, and only the doweled ends are reinforced. The panel sizes are designed to accommodate a 12 ft road width and only for half slab replacement. This design is not optimal for airfield pavements where more dowels are used to accommodate less channelized traffic. The dimensions limit repair versatility, since they do not align with typical rigid airfield pavement slab sizes.

The Air Force prototype system design consists of two completely doweled and reinforced transverse ends. A reinforcement grid is placed towards the depth of the panel to stiffen the slab and limit panel deflections when loaded by aircraft. The nominal side dimension of the square panels is 10 ft. The square panels yield potential for multiple installation configurations. The Air Force system is also supported by data from limited traffic testing completed on the prototype design, which shows it is capable of supporting one of the required aircraft types. Therefore, the Air Force prototype panel was selected as the most applicable for contingency airfield repairs.

Optimization of the selected precast panel system

The Air Force precast panel repair method had several drawbacks that required system modification or verification of specific design elements before field implementation in order to comply with AFCEC's requirements.

1. A review of typical airfield pavement slab sizes was required for determining the appropriate panel size. A single set of panel dimensions was needed in order to standardize fabrication methods and prefabricated, reusable formwork.
2. The reinforcement design must ensure the panels can withstand stresses encountered during all phases of the panel's lifecycle including lifting, transporting, and stockpiling without incurring damage.
3. The dimensions of the panels must allow for safe and efficient operation, while accounting for limitations in available lifting equipment at remote locations.

4. Streamlining personnel work tasks to fabricate panels in the field was required to ensure quality and performance. Adapting existing equipment or using common commercially available equipment and disposable supplies for both fabrication and field installation was needed in order to minimize training requirements.
5. Techniques for prompt demolition and removal of the existing pavement required exploration, since this was expected to be one of the most time consuming work tasks.
6. Specifications were needed for selecting effective backfill materials and expediting its placement as bedding material. Design strength requirements for this material should be based on preventing pavement damage under the various aircraft load configurations expected.
7. Leveling techniques needed to be refined to prevent having to extract installed panels and reseat them during the installation in order to meet surface requirements.
8. Rapid-setting cementitious materials placement methods needed to be developed to decrease the labor and time required to grout dowel receptacles.
9. Other work tasks required investigation to reduce the installation time and minimize airfield downtime.
10. The performance of the panels under C-17 traffic needed to be documented including failure modes, FOD potential, and tire hazards.
11. Both economic and utility analyses of the optimized precast system compared with other repair methods were needed.

4 Modifications to the Air Force Method of Precast Panel Repair

The selected Air Force prototype system was reviewed by ERDC and AFRL personnel to determine necessary modifications to meet the design requirements for contingency PCC repairs. The following items were identified as key component modifications detailed in this chapter:

- Acceptable panel dimensions
- Load transfer
- Multiple panel usage and configurations
- Formwork requirements for field construction
- Structural design verification
- PCC mixture requirements
- Lifting considerations
- Equipment and supplies requirements
- Opening to traffic time

Acceptable panel dimensions

Airfield pavement thickness and slab size varies from airfield to airfield. To develop a standardized system, a single panel size and thickness was investigated to allow for a variety of repairs on a typical airfield. The final panel size was controlled by the anticipated maximum equipment lifting capacity available for placing, transporting, and storing panels in a contingency environment.

Plan area

The original Air Force panel was designed for 10 ft x 10 ft slab replacements. Further refinement was required to allow for multiple installation configurations for larger slabs. To do this, plan area dimensions were selected that would facilitate development of a modular system that provided the capability to tailor the configuration of the precast panels to the repair locations.

The optimal method for modularity was to fabricate a set of formwork that allowed for casting custom sized panels for various sized slabs/repairs. Customization was determined to be too difficult in a contingency environment since unique panel designs would be required for each repair size needed. Additionally, construction materials and equipment needs would vary as the dimensions changed. For practical considerations and to limit the risk of failure from incorrectly constructed panels, a single set of precast panel dimensions for use on the entire airfield was determined to be the best course of action.

Panel dimensions

Military airfield pavement design guidance recommends joint spacings between 12.5 and 20 ft. Identifying a single panel size was difficult because the final dimensions must be versatile for all slab sizes that will possibly be encountered. Precast panel systems are typically thinner than the replaced pavement and have slightly smaller lengths and widths to allow ease of panel installation within the removed pavement void.

The initial Air Force panel dimensions were selected based on the slab size of the test section pavement used for field testing (10 ft x 10 ft x 1 ft). Panel dimensions were 9 ft 10.5 in. x 9 ft 10.5 in. x 11 in., which resulted in a 0.75-in.-wide perimeter construction joint around the panel. The precast panels were 1 in. thinner than the existing pavement to allow for bedding material to be placed on the subgrade to ensure uniform support under the precast panel. Trafficking installed panels showed the system was capable of supporting simulated aircraft operations (1,508 channelized passes of an 81,000 lb F-15E) with little distress (Ashtiani et al. 2010).

For the subsequent research phase presented in this report, the modified single panel dimensions measured 9 ft 11.25 in. x 9 ft 11.25 in. x 11 in. The resulting construction joint width was decreased to 0.375 in. to reduce the amount of materials required to seal the joints. Comparing the selected panel dimensions against the typical airfield slab sizes shows the greatest challenge the system will have is for a full slab replacement larger than 10 ft x 10 ft. Considerations to allow for connecting multiple smaller slabs to fully or partially replaced slabs up to 20 ft x 20 ft were needed.

Panel lifting operations

The dimensions of the panels were also dictated by the lifting capacity of the equipment available at most contingency locations. Assuming a PCC unit weight of 150 lb/ft^3, a 9 ft 11.25 in. x 9 ft 11.25 in x 11 in. panel weighed 13.6 kips. Many pieces of equipment could potentially be used lift these precast panels. However, the use of cranes was strongly recommended because their vertical lifting capability provides the safest operation when handling panels.

While forklifts may be useful for moving panels from the storage site to the installation site, their use was not recommended for installing precast panels. Traditional forklifts do not have the reach, and extendable boom forklifts do not have the capacity to safely install the panels. Exceeding the machine's capacity will lead to overturning, which endangers personnel, the forklift, and the panel. Additionally, front end loaders and excavators should not be used to lift panels during repair operations. These equipment types are not designed for vertical lifting. Therefore, an adequately rated crane is recommended for all lifting operations involving precast panels.

An investigation of anticipated lifting capabilities at contingency locations concluded that many bases would have access to at least a 15-ton crane. Smaller cranes would not be suitable for lifting the 13.6 kip panels. The major limitation of this capacity crane is its small lifting radii. Small lifting radii require the crane to be situated very close to the repair area. In order to place multiple panels, the crane must be moved between panel installations. At minimum, a 15-ton crane is recommended for precast panel repair operations, but higher capacity cranes should be used if available to reduce the required number of crane set-ups for multiple panel installations. A 30-ton crane is recommended to complete a multiple panel replacement in one setup location.

Construction joint consideration

In designing the construction joint, the ability to install the panel within a saw-cut repair boundary was also considered. The panel must be slightly smaller than the repair area to allow the panel to be installed easily with little resistance around the cut perimeter and to allow for any variation in the alignment of the edge from the saw cutting. However, the gap between the replacement panel and existing pavement could not be so wide that a significant volume was not filled by the repair panel, creating a FOD/incompressible material trap. Additionally, wider joints have been proven to reduce the load transfer efficiency of the repair, which can result in spalling or faulting at the joint.

The original Air Force panel design used 0.75-in.-wide joints that were backfilled with a rapid-setting rigid pavement repair material. To reduce the amount of backfilling required and FOD potential, the modified panel plan area dimensions were altered to 9 ft 11.25 in. x 9 ft 11.25 in., and the construction joint was reduced to 0.375 in. This was deemed to be the smallest practical joint to provide enough tolerance to account for potential saw kerf issues and precast panel dimension irregularities.

Backfilling the joint with a rapid-setting repair material causes concerns of FOD generation since the panel has no ability to expand thermally without bearing on the repair material. Also, the panels were expected to be thinner than the existing pavement and would deflect slightly more under loading than the parent pavement. Both cases change the loading that the joint repair material experiences and could be damaging to the joint repair material.

One item missing from the original Air Force design was joint sealant around the perimeter of the repair. In addition to preventing water from infiltrating below the slabs, joint sealant prevents FOD and other incompressible material from collecting in the unsealed joint. Since controlling FOD is critical on airfields, airfield grade silicone joint sealant with foam backer rod was recommended as an added step in the repair method. Silicone sealant was selected since it can be applied cold and requires no preparation time. HDPE tape or backer rod was required for field forming joints to prevent unfavorable sealant bonding conditions that decrease performance and to control the cross-sectional dimensions (mainly depth). A closed cell, polyethylene backer rod was chosen for its ease of use and ability to minimize sealant losses if slightly oversized compared to a thinner and incompressible tape. Products chosen for installation were recommended to follow those given in Unified Facilities Guide Specification (UFGS) 32.13.11: Concrete Pavement for Airfields and Other Heavy Duty Pavements.

Load transfer

When an aircraft passes over a joint in concrete pavement, the slabs on each side of the joint deform because of load transfer between the slabs. Load transfer efficiency (LTE) is a key design parameter that greatly influences the nature of load distribution across the concrete slabs. Failure to properly characterize the load transfer devices significantly jeopardizes the efficiency of a repair using precast panels. This can manifest itself by initiation and propagation of micro-cracks across the loaded and unloaded slabs. Load transfer dowels are one mechanism utilized to develop adequate LTE.

Deflection based LTE collected using HWD joint tests is the most common method to calculate LTE. Equation 1 shows how deflection based LTE is calculated. The acceptance threshold for LTE_{δ} is designated as 0.7 (or 70 percent) for joints perpendicular (transverse joints) to the direction of traffic. There is no load transfer threshold requirement for joints parallel to the direction of traffic.

$$LTE_{\delta} = \frac{d_u}{d_l} \tag{1}$$

where

LTE_{δ} = deflection based LTE
d_u = measured deflection under unloaded slab
d_l = measured deflection under loaded slab.

The original Air Force panels utilized load transfer dowels along each transverse edge (perpendicular to traffic) of the panel, situated such that the dowels were parallel to the direction of traffic. No load transfer dowels were installed along either longitudinal edge. Installed panels were trafficked with an F-15E load cart, and nondestructive testing using an HWD was conducted at defined traffic intervals to calculate load transfer efficiency along both the transverse (doweled) and longitudinal (undoweled) edges. LTE along the doweled edges was determined to be approximately 0.95, well above the threshold. Although not required, an LTE analysis was also conducted along the undoweled longitudinal edges. LTE in this direction was determined to range from 0.70 to 0.85; therefore, meeting the transverse joint threshold criteria despite the absence of load transfer dowels along those edges (Ashtiani et al. 2010).

A three-dimensional finite element (FE) analysis was also conducted to determine if increasing the LTE along the longitudinal edges, by adding load transfer dowels in that direction, would improve the performance of the precast panels. The analysis showed that increasing the LTE to 0.95 did not significantly enhance precast panel performance in response to loading (ARA internal communication 2011).

Modification for multiple panel usage

For the precast pavement system to be effectively used for larger airfield slabs, the system must allow for connecting multiple panels to repair larger areas. The original design did not allow for connection of multiple panels, thus an additional panel was created to work in conjunction with the modified single panel.

Installation configurations

Three definitions of the repair configuration were established to categorize the repairs that could be made with the modified system. These definitions are provided below, and Figure 11 details the repair types visually for clarity. Nonstandard configurations utilizing panels placed away from the corners of a damaged slab, like those detailed in Figure 12, were not considered for implementation since there are no traffic testing data available regarding their performance. Their use is not recommended until performance data is gathered since these repairs do not maintain the existing joint plan and would not be supported by current UFC guidance.

- Single panel: Removal of one quarter or more of the existing pavement slab followed by installation of a single precast panel.
- Double panel: Removal of half or more of the existing pavement slab followed by installation of two precast panels.
- Quad panel: Removal of the entire existing pavement slab followed by installation of four precast panels.

Panel types

To allow for connecting multiple panels together, the Air Force system panels were reconfigured to consist of the original *standard panel* and a new *terminal panel*. Both panel types are shown in Figure 13. The absence of load transfer dowels along the longitudinal edges of both panel types should be noted. As mentioned previously, FE analysis of the original Air Force design showed that the addition of dowels on the longitudinal edges would not significantly enhance the performance of the installed panels.

The original Air Force system panel design was maintained for the modified system and labeled the *standard panel*. The panel had dowels provided at the mid-depth of both transverse edges. A minimum of one standard panel is required for each repair made in any configuration and must be used to complete any repair where multiple panels are used together.

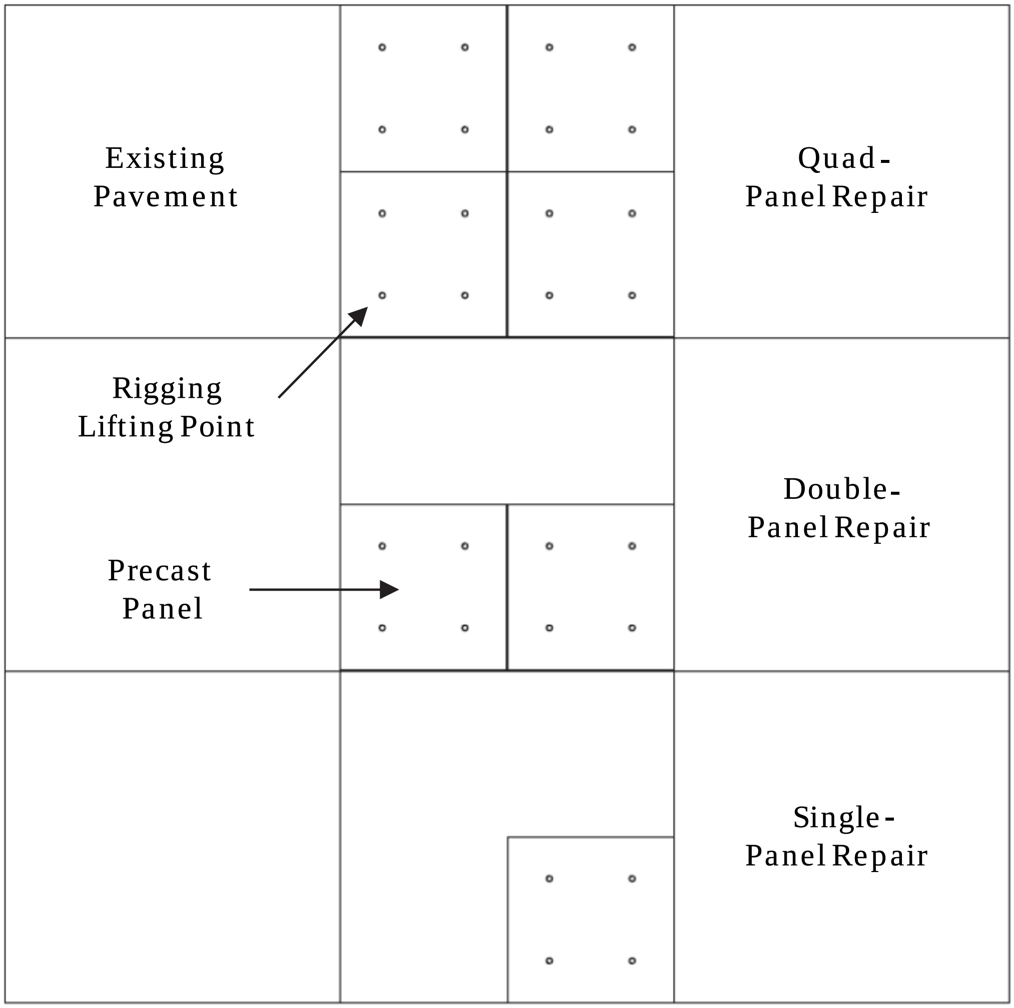

Figure 11. Corner installation configurations.

The difficulty with connecting multiple panels using this panel type alone was the dowels lack an installation location within the panel. When a single panel was connected to the existing pavement, a female slot was chiseled into the adjacent pavement to produce a dowel receptacle or slot. A rapid-setting repair material was then used to fill the receptacle and embed the dowel into the parent PCC. The biggest challenge when connecting panels was constructing the receptacles in the panel for the dowels. Simply removing the dowels on one side of a standard panel did not prepare that side to accept dowels. Saw cutting the precast panels and chiseling out dowel receptacles would damage the internal reinforcement, and it would not function as designed.

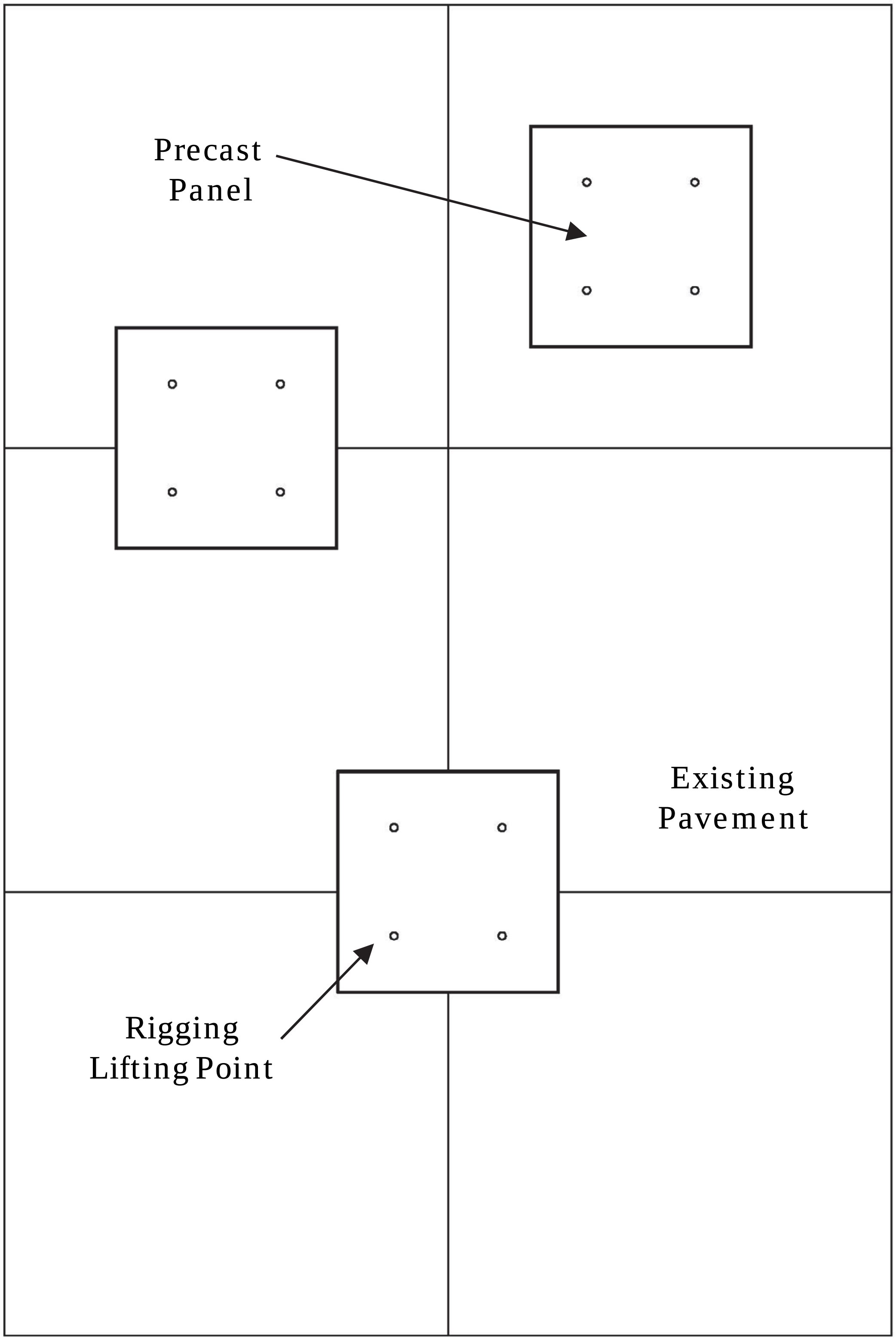

Figure 12. Nonstandard configurations.

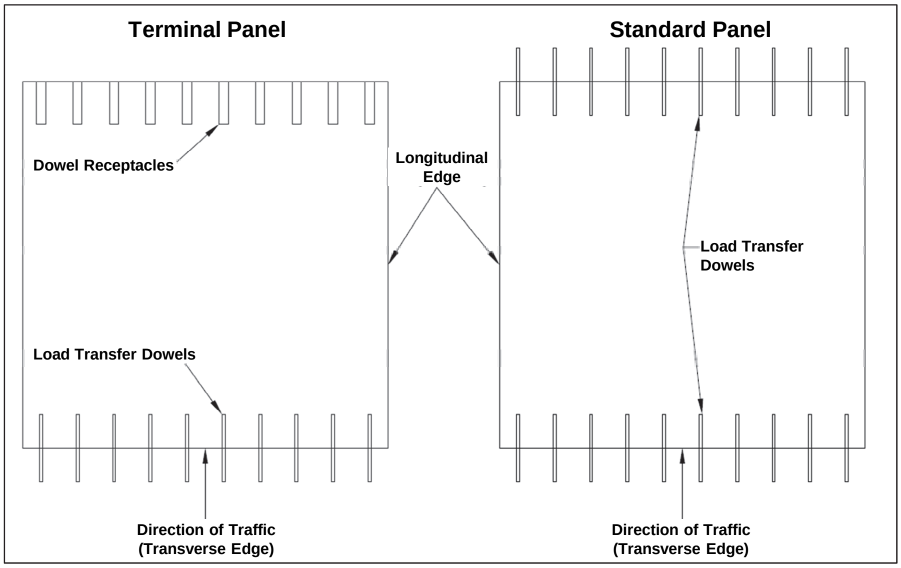

Figure 13. Precast concrete panel types.

Consequently, a second panel, called a *terminal panel*, was designed for connected panel repairs only. One transverse edge had the same design as a doweled standard panel transverse edge; however, the other transverse edge included female dowel receiving receptacles. The internal reinforcement was moved into the interior of the panels and was supported by reinforcement chairs. Additional information on the final positioning of reinforcement is presented later in this chapter in the structural design verification section.

The precast receptacle construction technique will be discussed in more detail later in this chapter. Initial receptacle molds were constructed out of wood suspended over the panels when casting; however, these were modified to a steel mold bolted onto the formwork to allow for reuse and to reduce the construction time required to make disposable wooden molds.

Formwork for field construction

Concrete is heavy and exerts significant loads on the face of the forms. Formwork must be carefully designed and constructed to ensure the dimensions of a precast product meet panel design tolerances and that the forms can withstand repeated use. Traditionally, this is done with steel or wooden forms, ranging in complexity from simple sections to customized designs.

Old formwork

The formwork used to fabricate the original Air Force panels was constructed with steel channel sections (C12 x 30). Several precast panels were cast with these forms, and there was no deflection evident with the formwork or bowing measured in the fabricated panels. However, the weight of the forms made placement and maintaining formwork squareness difficult. The updated formwork was constructed with a lighter steel channel section (C12 x 25) bolted together at butted, keyed connections (Figure 14). This version of the formwork was used for this subsequent research phase. A 1-in.-thick layer of plywood or oriented strand board (OSB) was placed within the formwork to provide a false bottom and yield an 11 in. panel depth. The transverse form pieces used an additional piece of square tube (HSS 2 x 2 x 1/8) to support the dowel bars after installation.

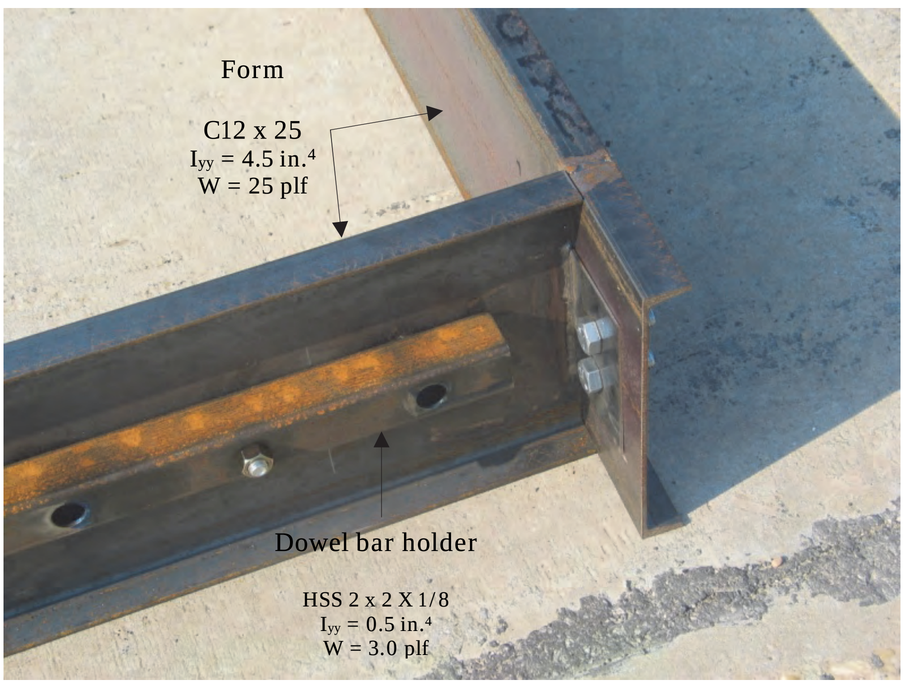

Figure 14. Modified Air Force formwork components and features.

The formwork was further modified before casting the panels for field testing to decrease bowing of the formwork after a precast panel training demonstration conducted at the ERDC in July 2011 revealed bowing of the forms. This bowing resulted in panels with slightly trapezoidal panel cross-sections (Figure 15). The shape resulted from the top of the form rotating outward about the fixed base (by a concrete expansion anchor). An additional square tube (HSS 2 x 2 x 1/8) section was welded to the mid-height of the longitudinal formwork to add stiffness.

Figure 15. Dimensional problems with first precast panels.

Design of modified formwork

Since the tested formwork was not capable of providing the desired panel shape and dimension, the formwork was redesigned. Bearing pressure on formwork from placed concrete acts as a distributed load vertically (in-plane) and horizontally (out-of-plane) per unit length along the face; however, the amount and modeling of load applied differs. The distributed load is a function of its depth from the surface and the unit weight of the material used. This load is constant along the length of the face. The loadings can be modeled as triangular and rectangular distributed loads, respectively.

Applying this loading scenario to the formwork design, the single, complex three-dimensional loading was converted into two simpler, two-dimensional situations where simple beam theory could be used. The resultant triangular load from the vertical formwork pressure was determined, and the resulting force of the triangular load was applied as a rectangular distributed load across the length of the form. Figure 16 details the concept and shows calculations for determining the required values for the beam theory calculations.

With the loading situation modeled, the values determined can be applied to beam deflection equations to size the formwork. With a user defined maximum allowable deflection and formwork material stiffness (modulus of elasticity), the minimum amount of geometric stiffness required for the formwork cross-section (moment of inertia) was determined. The models considered are for a simple beam with two different support conditions. Equation 2 models a beam between fixed, very rigid supports (as depicted in Figure 16). Equation 3 models the beam between simple supports that offer no resistance and is more flexible. The actual support conditions are somewhere in between since the connections are somewhat flexible; however, the exact support condition is not known.

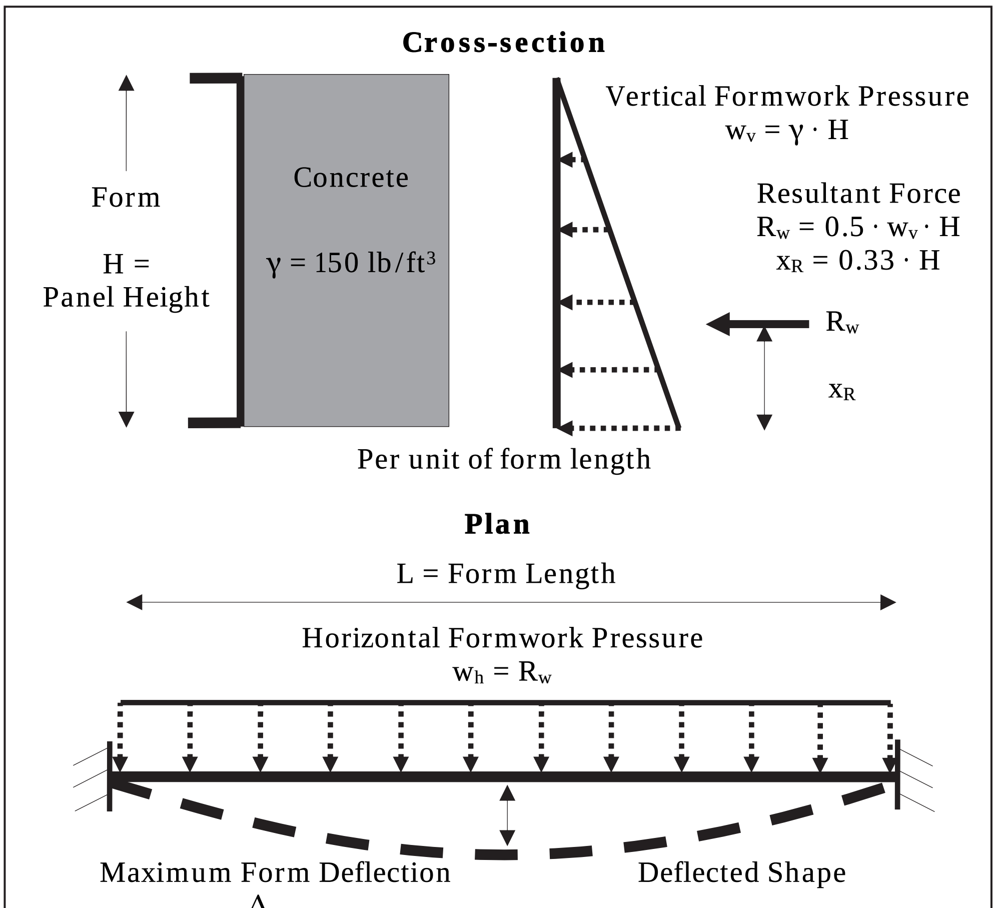

Figure 16. Free body diagram of loading on formwork.

The minimum moments of inertia required for each model were 1.50 and 7.50 in.^{4},with steel forms and a maximum deflection of 0.0625 in., respectively. Examination of the equations shows that Equation 3 was more stringent requiring a stiffer cross section to be used. Results from Equation 3 were used since they were the most conservative even though this will result in heavier formwork.

$$\Delta_{\max} = \frac{qL^4}{384EI} \rightarrow I_{\min} = \frac{R_w L^4}{384E\Delta_{\max}} \quad (2)$$

$$\Delta_{\max} = \frac{5qL^4}{384EI} \rightarrow I_{\min} = \frac{5R_w L^4}{384E\Delta_{\max}} \quad (3)$$

where

E = material modulus of elasticity, 29 x 10^6 lb/in.^{2} for steel
I = cross section moment of inertia, in.^{4}
Δ_{max} = maximum deflection, in.
q = distributed load, lb/in.
L = length, in.
R_w = resultant force, lb/in.

The situation modeled in Figure 16 does not precisely model the boundary conditions of the formwork. Installation of the concrete anchors into the pavement at the midpoint of the form fixes the base of the form at this location, and the deflected shape will not truly resemble that as shown. However, in contingency situations, a concrete surface may not be available for anchoring the formwork. In these situations, the expansion anchors cannot be used. Other reasonable casting surfaces, like asphalt pavement or compacted gravel may be encountered, and the formwork should be versatile enough to accommodate this scenario. The minimum section requirements were based on accommodating this worst-case casting scenario.

Use of 0.0625 in. for the maximum deflection of the formwork was considered a reasonably achievable value that would still allow for erection of the formwork by hand. Decreasing the maximum deflection required a stiffer section with a larger moment of inertia. Larger moments of inertia typically require sections with thicker walls or larger cross-sections. This increases the overall weight and dimensions, making the form more unwieldy to handle by personnel. Efficient selection of the smallest section was required to ensure the formwork could still be manipulated without heavy equipment.

Upon review of the moments of inertia, researchers determined the modified Air Force designed formwork cross-section (C12 x 25) was under-designed. The new design required the selection of a steel section that could easily form the vertical face of the panel while providing a horizontal surface for screeding the panel's surface. Review of commercially available steel sections showed that only channel (C designation), miscellaneous channel (MC designation), or hollow structural sections (HSS designation) fit these criteria.

Examination of typical C, MC and HSS sections showed that a section height of 11 in. was not available, and section heights of 10 or 12 in. tall were the closest commercially available sections. To produce 11 in. thick slabs, two options were available. The first option was to use a false bottom technique utilizing a 1-in.-thick layer of plywood used in the original Air Force design, allowing a 12 in. section to be used. The second option required a custom built-up section designed using smaller commercially available component sections. The built-up section was selected as the best option, as it minimized the use of construction materials.

As previously stated, built-up form sections are made up of smaller components combined to achieve the composite properties required. The formwork cross-section design focused on connecting multiple commercially available HSS sections together for economy and ease of constructability. Constructing a fully custom beam from plate steel would minimize the cross-section's constructed weight, but the added fabrication cost of cutting and connecting the various plates together would make this option cost prohibitive.

Pre-manufactured C and MC sections were not considered for the design. The C sections are inefficient shapes for this formwork because significant stiffening is required to meet the minimum moment of inertia requirements. Looking through all candidate C sections showed the largest 10-in. tall section had only about 60 percent of the minimum moment of inertia needed. The MC sections are stiffer than equivalent C sections; however, the weight of the minimal section meeting the requirements was significantly higher than the minimal HSS section needed.

The general design of the formwork consisted of 3 simple parts. The main structure consisted of a large, single HSS section that met the minimal moment of inertia requirement. A small filler piece was welded to the top to achieve the total composite height of 11 in. Selection of the tallest section available for the main structural unit was most efficient because it minimizes the total weight, ensures maximum geometric stiffness with the least amount of components, and provides no clearance issues with any accessories installed within the interior of main structure. A thin piece of plate or sheet was welded over the form face to cover the rounded corners of the rolled HSS sections and provided a clean, vertical face.

Figure 17 shows the minimal section requirements and assembly for the modified formwork. The main structure component selected for construction was based on its height, local shape availability and ability to meet the minimum 7.5 in.^{4} moment of inertia requirement. Only the area portion of the moment of inertia values for each section shape was considered since the centroid was fairly centered in the horizontal direction. Any additional moment of inertia created from the parallel axis theorem portion of calculations not completed was negligible since the main structure dominated the calculation and only assisted the design by making the composite shape stiffer.

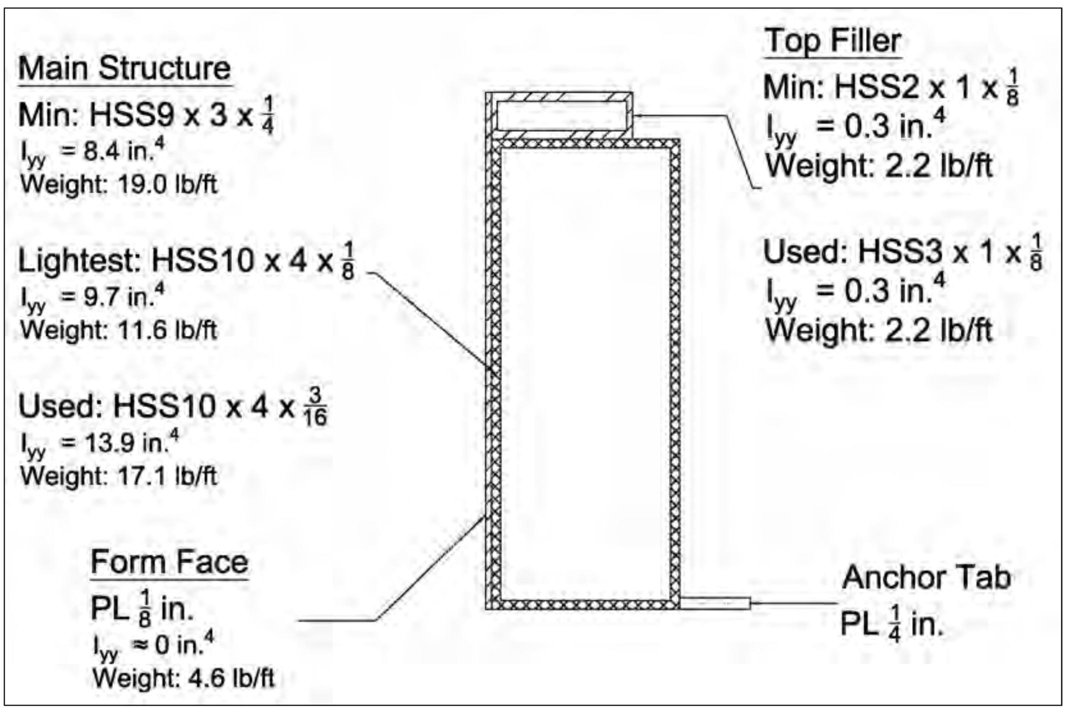

Figure 17. Modified formwork nominal cross-section requirements and prototype construction.

Alterations to the main structure's width and thickness dimensions were expected based on economics and locally available material. When ERDC constructed a set of formwork following the minimum values, the wall thickness and dimensions were increased to use locally available HSS 10 x

4 x 3/16. Care should be taken not to excessively increase dimensions past those required to allow for the forms to be put together by hand.

Cases may arise where it becomes necessary to tailor the height of the main structure member. A review of sections shorter than 10 in. was completed to determine their adequacy for use. The smallest allowable height for this member based on the commercially available sections that provide the minimum moment of inertia, acceptable corresponding top filler piece selection, and positioning of formwork accessory elements discussed later in this chapter was 9 in. Some accessory items will need minor repositioning to be used as intended at their intended locations.

Load transfer device installation

Specific issues with the way the load transfer devices were cast into the panels were identified during the field demonstration of the original formwork. Significant changes were made to allow for easier, more efficient construction of these items.

Male ends

The original Air Force system cast dowel bars into the panels during panel fabrication. Plain steel dowel bars were placed into the fresh concrete and were held in place by a removable holder affixed to the formwork. After the concrete hardened, the holder and formwork were removed over the fixed dowel. Figures 18 through 20 detail the removal process for the equipment and formwork. If the formwork could not be removed easily, the dowels were removed by rotating them with a 24-in. pipe wrench to break the bond with the concrete. Vigorous back and forth rotation was required to remove the dowels.

Following review of this process, plain steel dowels were considered unacceptable from a corrosion standpoint. Since the panels will most likely be stockpiled in outdoor locations, epoxy coated steel dowels were recommended. Modifications to the formwork were made to accommodate the coated dowels and to prevent damage to the factory applied epoxy coating during fabrication. The epoxy coating thickness added an additional 0.03125 in. to the diameter of the dowel bar that must be accounted for in the formwork design.

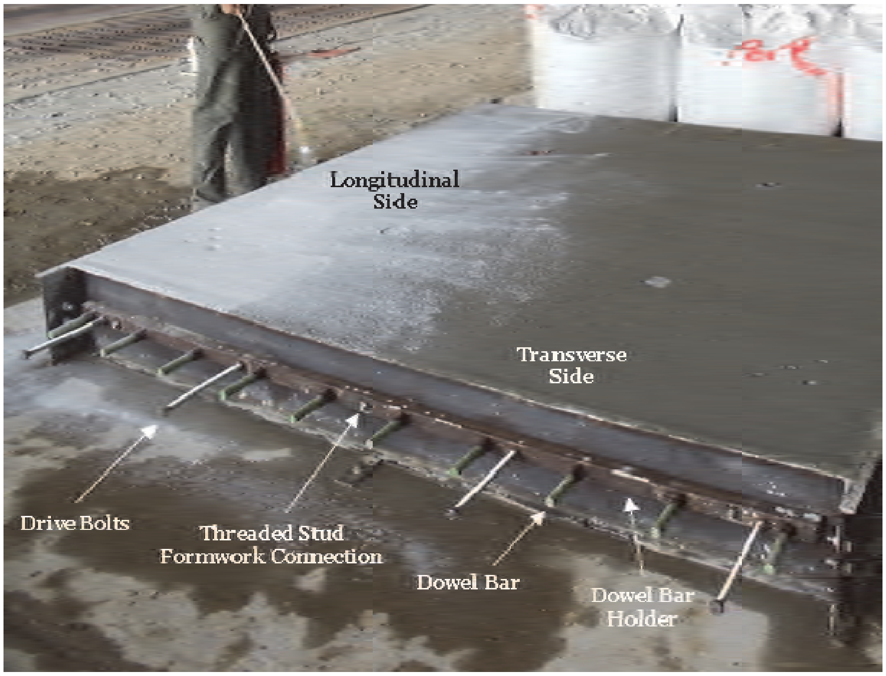

Figure 18. Wet-set dowel installation equipment.

Figure 19. Removal of dowel bar holder.

Figure 20. Transverse form removal process.

Following review, the formwork removal process was believed to be too labor intensive and damaging to the panel. Formwork removal timing for the transverse sides with dowels was over 30 minutes per side compared to less than 5 minutes for the longitudinal side. The large disparity in time resulted from the dowels protruding through the formwork. It was difficult to remove the dowel bar holder that spanned the length of the form and equally difficult to slide the formwork off the dowels. Not having a well-placed point for the pry bar to bear on without damaging the panel added to the difficulty and time required for removal.

The dowel bar holder was redesigned to address the observations previously described. Figure 21 details the design of the modified holder. Individual holders replaced the long, single piece unit to allow for even removal across each dowel. Each holder consisted of two separate pieces. The front plate consisted of a hollow tube welded to a thick steel plate. The back plate was a similarly sized steel plate welded to the back of the formwork and was used to hold the front plate in position. The tube allowed for accurate and easy placement of the dowel through the wide formwork when casting.

After the placed panel concrete cured, each dowel bar holder was removed (leaving the dowel in place). The process of using bolts to assist with removing the holder was maintained but was applied to each individual

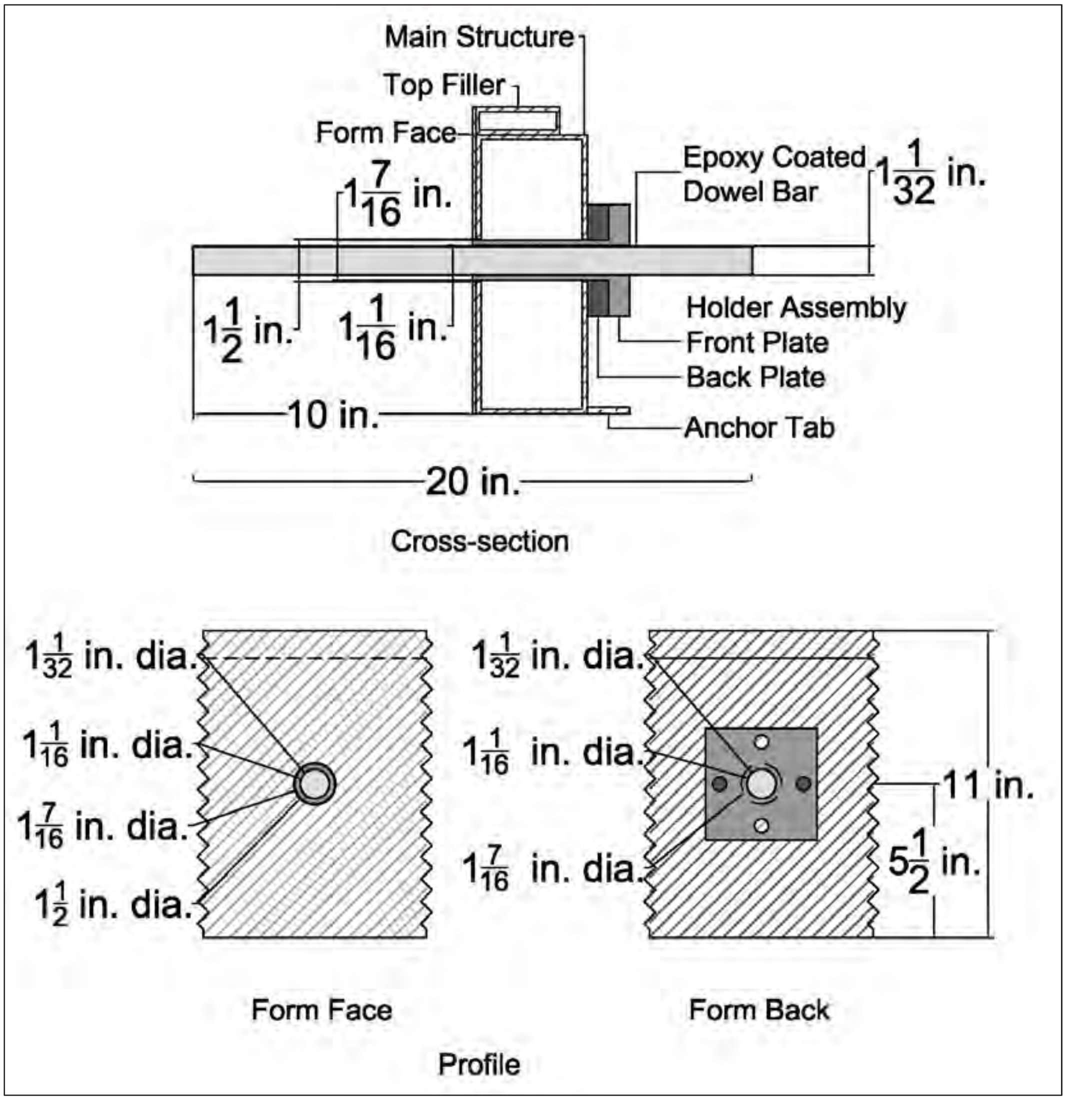

Figure 21. Dowel holder concept drawing.

holder. Two bolts were driven through tapped holes on the front plate to push the front plate away from the back plate for its removal. The thickness of back plate and the dowel fixed into the hardened panel provided a significant increase in surface area used to guide the dowel holder off the dowel compared to only the thin wall thickness of the main structure of the formwork (or thin web from the original formwork).

Removing the larger tube provided an oversized hole in the formwork that reduced the risk of dowel damage caused by uneven sliding of the formwork off the dowels. Subsequent trial panel fabrications revealed little to no prying of the formwork was needed with the oversized hole in the formwork as well as reducing potential damage to the panel.

Dowel receptacles

The initial dowel receptacle design used wooden boxes suspended into the fresh concrete. The boxes were attached with screws to lumber to obtain proper spacing. After the concrete hardened and the steel formwork was disassembled, the wooden boxes were removed. New sets of boxes were required for each terminal panel cast because they could not be salvaged after demolition. Figure 22 shows the wooden formwork erected for casting dowel receptacles.

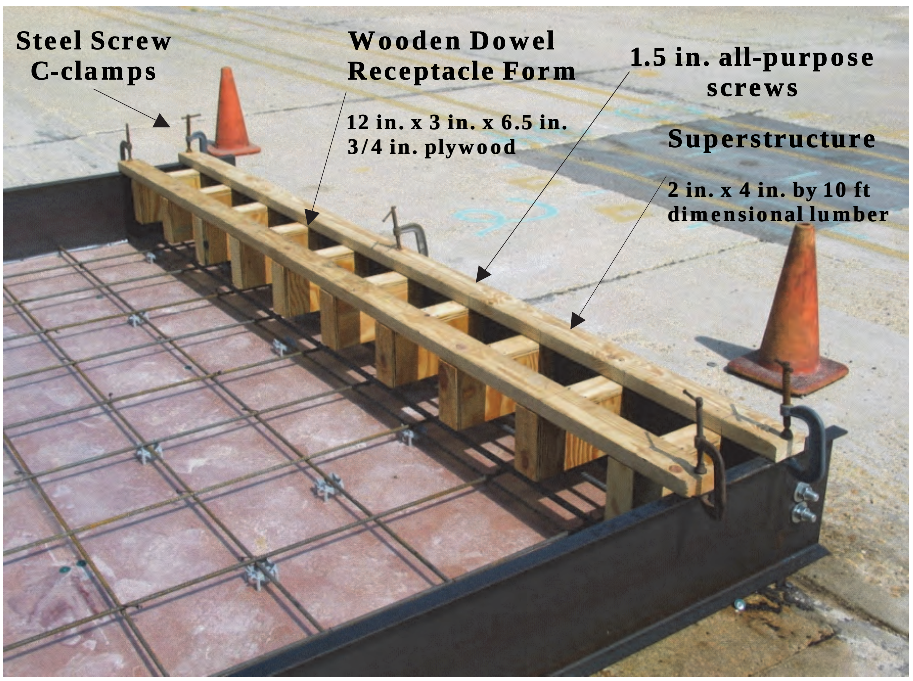

Figure 22. Original dowel receptacle casting construction.

Multiple issues with the wooden box method were noted including the materials used, the amount of accuracy required, and the potential for misalignment in the field. Limiting the complexity of the design was given utmost consideration. Reusable, easy to use supplies and equipment were selected to minimize variations between panels.

Placing the dowel receptacle on the surface of the panel made field installation significantly easier and reduced equipment needs. Consideration was given to constructing a receptacle on the underside of the panel, as other systems use this design to allow for cleaner, smooth pavement surface; however, these methods require pressure injecting grout from the surface to fill the receptacle void. Without performance data showing casting the dowel receptacle on the surface is detrimental to the panel's operation, casting receptacles from the top was decided the best option for ease of construction and reduced equipment needs.

Researchers redesigned the receptacle form to allow reuse. Figure 23 details the construction of the modified form. Hollow, interchangeable, steel boxes were selected since they can be reused if cleaned and maintained properly. The height of the box was selected to provide the minimum ½ in. clearance between embedded steel and concrete as recommended in UFC concrete repair guidance. The boxes mounted flush to the interior face of the formwork. Bolts held the form in place. A slight trapezoidal cross section was made to allow for easy removal and minimize damage to the panel.

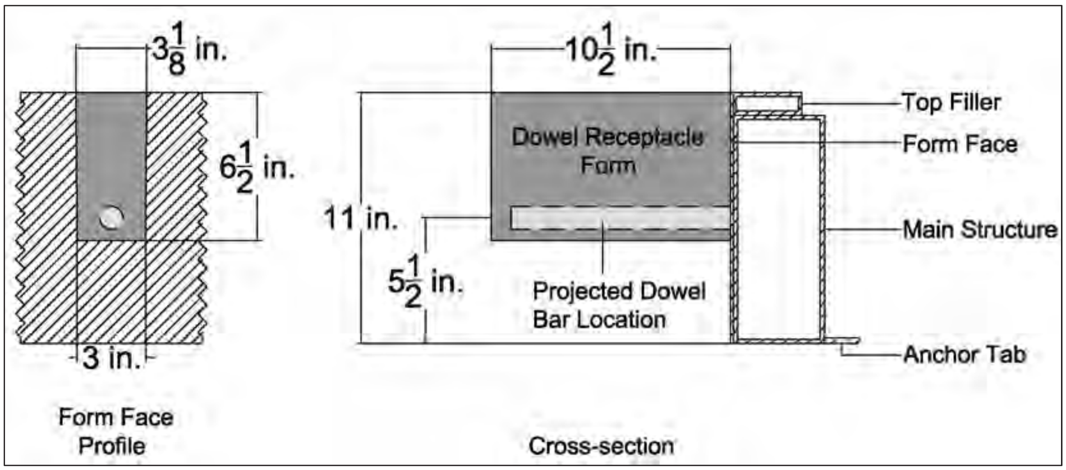

Figure 23. Revised dowel receptacle casting construction.

Early prototypes of the steel boxes worked well, but there were challenges when fabricating the boxes with steel C sections. Shipping or fabrication defects in the formwork caused the boxes to not bear flush against the form face. Metal shims were required to correct this; however, this made each receptacle form customized to a specific location on the formwork preventing interchangeably. Lack of straightness is not expected to be an issue when using HSS steel sections since ASTM International (ASTM) A500 has a limit of ¼ in. (for 10 ft lengths) for straightness when produced, compared to no limit for sweep (out of plane curvature) provided for rolled steel shapes in ASTM A6.

Formwork connections

The formwork was designed to be broken into pieces to allow for easier handling and packaging as a kit. Two bolted connections were used at different locations to construct the square perimeter of the panel as shown in Figure 24.

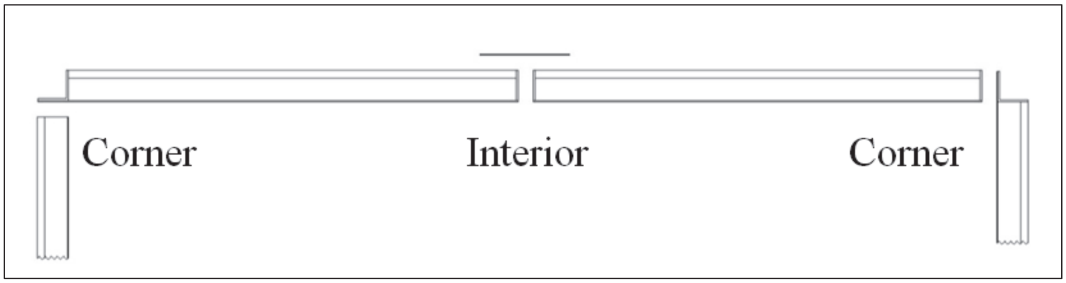

Figure 24. Formwork connection types and erection plan.

Corner

The bolted corner connection was maintained from the original system design to allow for easy form construction; however, the connection mechanism was modified for simpler erection. Figures 25 and 26 show the original connection design. A keyed butt joint was used between the slotted transverse and keyed longitudinal sides to correctly position the formwork. A thin tab extended off the transverse side to clamp the pieces together. The original drawings specified a 1/8 in. plate be used for this tab; however, a thinner 1/16 in. sheet was used on the formwork provided to the ERDC. The longitudinal side was made slightly longer than the transverse span to accommodate the tab.

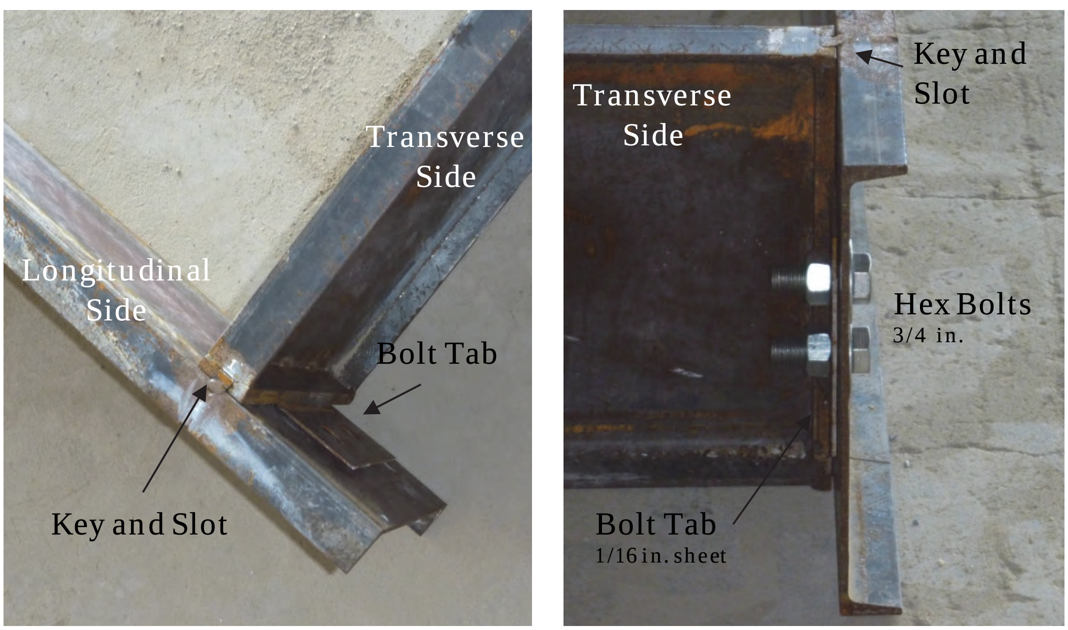

Figure 25. Original corner connection construction.

Figure 26. Close-up of keyed connection.

Modifications focused on stiffening the corner for constructability purposes and to ensure that corners were square. Although the keyed butt joint was effective at positioning the formwork and was very simple to put together, its short channel flange width, slenderness of the keyway components, and thin bolt tab were not sufficient to resist rotation. The keyway was removed since it offered little structural support and made formwork fabrication difficult compared to a bolted connection. The sheet metal bolt tab was also changed to a thicker square angle (L 4 x 4 x ¼) welded to the end of the form. Integrating the bolt installation points of the connection into the cross-section allowed for the removal of 6 in. of form length from the longitudinal pieces for a lighter and smaller sized form. An additional bolt was added to the design, and all bolt locations were spread out along the connection to ensure uniform bearing along the formwork height. The new corner design is presented in Figure 27.

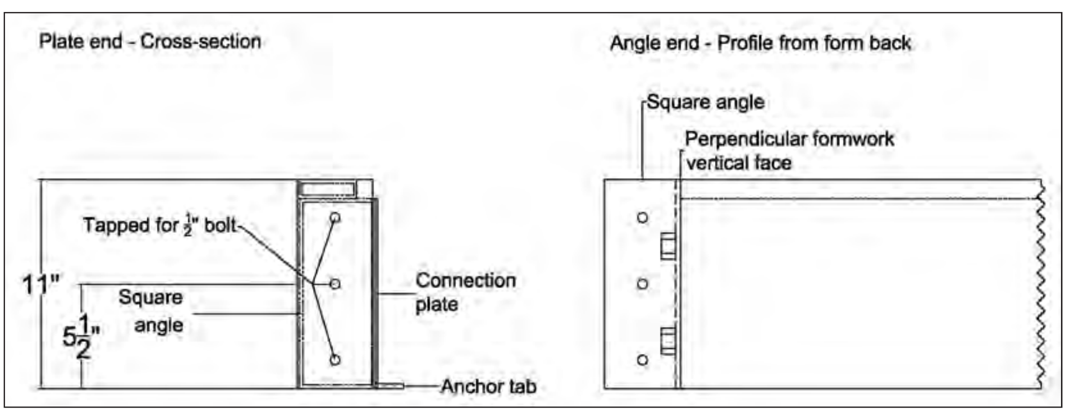

Figure 27. Corner connection concept drawing.

Interior

One concept not explored in great detail with the previous system was the mobility of the formwork. The Air Force wanted to arrange a storage container based kit with all the supplies and small equipment required to construct and install panels on site at remote locations. The dimensions of the formwork used should allow for efficient storage within the storage container's limited dimensions.

The main issue with the formwork was its irregular size and weight compared to other items considered for inclusion in a kit. The forms were long, thin and heavy. The thin cross-section of the formwork was not a significant issue and did not limit its storage location within a container; however, the length and the weight were far more challenging. The length of the form limited its storage to parallel with the long dimension of a standard military shipping container. Standard doorway dimensions of 8 ft x 8 ft x 10 ft were not able to accommodate 10 ft nominal length formwork. Storage along the diagonal direction was not efficient, and the weight of the formwork forced its storage to lower levels (floor) for container stability and retrieval.

As a result of the packaging challenges, the formwork was broken up into two pieces. Figure 28 details the modified formwork construction. Cutting the formwork in half allowed it to be stored in any direction within the container. The weight of the pieces, while approximately half the weight of the entire length, were still relatively heavy and required placement on or near the floor of the container.

The challenge with breaking the formwork into pieces was constructing an effective mechanism to reconnect the pieces. The connection must allow the completed formwork to perform as if uncut. The researchers believed the best approach was to have a two-step connection process. The first step would bring the two opposing pieces of formwork tightly together axially to ensure the correct length of the formwork. The second step would complete the connection by attaching a simple plate to the back side of the form to restore the structural rigidity of the beam.

The formwork required cutting in different locations for the longitudinal and transverse sections due to dowel bar installation. Longitudinal sides of the forms were cut in the center to divide the weight between each piece as close to equal as possible. The dowel spacing prevented breaking the

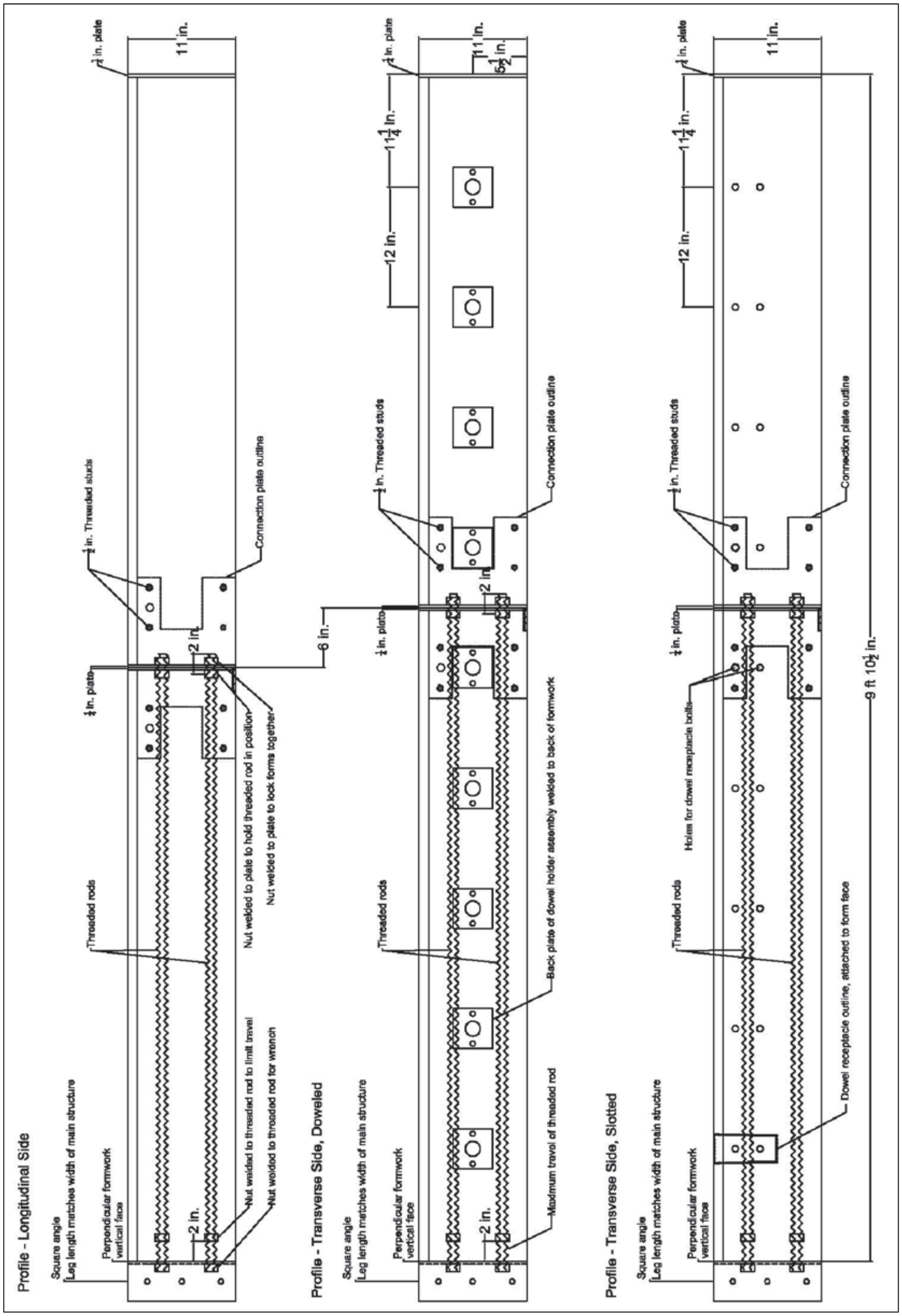

Figure 28. Interior connection mechanism concept drawing.

transverse formwork at the center of the span because a dowel was positioned at this location. The break was instead made 6 in. on center away from the central dowel.

The research team believed a screw mechanism was the best option for the first step of the interior connection based on its simplicity. Dual threaded rods were driven through tapped end plates to draw opposing sides together. The mechanism components were placed down the center of the main structure's hollow interior to protect it from damage and concrete buildup. Since the rods could not be accessed easily, the travel of the rods was limited to prevent it from coming out of alignment.

A connection plate across the interior connection was required to resist the maximum bending moment that the threaded rods could not support alone. This plate was designed to bend about the formwork and match the formwork's curvature. The minimum plate thickness was determined using the moment-curvature relationship from beam theory (Equation 4). Additional supporting equations were required for this analysis and were documented in ERDC's design calculations.

The following summarizes the supporting work required to use Equation 4:

- The radius of curvature at the form back of the solid (uncut) formwork was calculated.
    - Calculations involved using the flexure formula and Hooke's Law (1-dimensional) to find the longitudinal normal strain at the tension face of the formwork (form back) when loaded.
        - The out of plane moment of inertia for the minimum sized steel sections allowed for the built-up formwork cross-section was computed and 12.3 $in.^4$ was determined. This value was slightly higher than expected because the corners of the rectangular tubes were calculated as square and not as fillets. The nominal weight of the built up section was 26.6 lb/ft.
        - The maximum bending moment of the formwork is at the center of the span.
    - The radius of curvature for the formwork was found using the relationship between strain and curvature.

- To restore structural rigidity, the connection plate's radius of curvature computed was the value determined for the solid formwork plus half the thickness of the plate.
- Simplifying and rearranging Equation 4 yielded a single equation with two unknown variables. However, since the dimensions of the built-up cross-section were known, only the plate thickness variable remained.

$$k = \frac{1}{r} = \frac{M}{EI} \rightarrow I_{\min} = \frac{Mr}{E} \rightarrow 1.025 + 2.44 \bullet 10^{-4} t \\ \rightarrow I_{\min} - 2.44 \bullet 10^{-4} t = 1.025 \tag{4}$$

where

κ = Curvature, in.$^{-1}$
ρ = Radius of curvature, in.
I = Cross-section moment of inertia, in.^{4}
E = Modulus of elasticity, $29 \cdot 10^6$ lb/in.^{2} for steel.
t = Plate thickness, in.

Further reduction or simplification of Equation 4 was not practical to complete by hand due to the computational effort and risk of errors. To solve for the minimal connection plate thickness, a spreadsheet was made to calculate the moment of inertia the cut formwork had with various plate thicknesses. Table 5 documents the commercially-available plate thicknesses considered and the connection's resulting cross-sectional properties. The load bearing moment of inertia determined was a combination of the connection plate cut to the height of the structure's main structure and the threaded rods used for the first part of the connection. The optimal plate thickness selected after completing computations for the various plate thicknesses was the one that minimized the difference between both sides of Equation 4. Negative difference values found when making comparisons on Equation 4 showed the plate was under designed.

Calculations showed that 13 gauge sheet steel provided the minimum thickness required for the connection plate; however, 1/8 in. plate steel was selected for use because other formwork components use this material. Plate dimensions and connection details are shown in Figure 29. Plates were installed over the center of the interior connection and bolted to the formwork with 0.5-in.-diameter threaded studs extending from the formwork. Custom cut plates were required to use on all three formwork

Table 5. Connection plate thickness optimization.

Plate				Calculated Section Properties		Comparison against Equation 4	
Description	Thickness, in.	Weight, lb/ft	Plate's distance from form face, ft	Centroid distance from form face, ft	I_{yy}, in.^{4}	Left side	Difference between right and left sides
1 in. plate	1.00	30.6	3.63	3.45	4.00	4.00	2.97
½ in. plate	0.50	15.3	3.38	3.09	2.39	2.39	1.36
1/4 in. plate	0.25	7.7	3.25	2.79	1.72	1.72	0.69
3/16 in. plate	0.19	5.7	3.22	2.67	1.51	1.51	0.48
1/8 in. plate	0.13	3.8	3.19	2.50	1.24	1.24	0.22
11 gauge sheet	0.12	3.7	3.18	2.48	1.21	1.21	0.19
12 gauge sheet	0.11	3.2	3.18	2.43	1.13	1.13	0.10
13 gauge sheet	0.09	2.8	3.17	2.36	1.04	1.04	0.01
14 gauge sheet	0.08	2.3	3.16	2.29	0.93	0.93	-0.09
16 gauge sheet	0.06	1.8	3.15	2.20	0.81	0.81	-0.21
Threaded Rod Only	-	0	-	1.63	0.03	0.03	-0.99

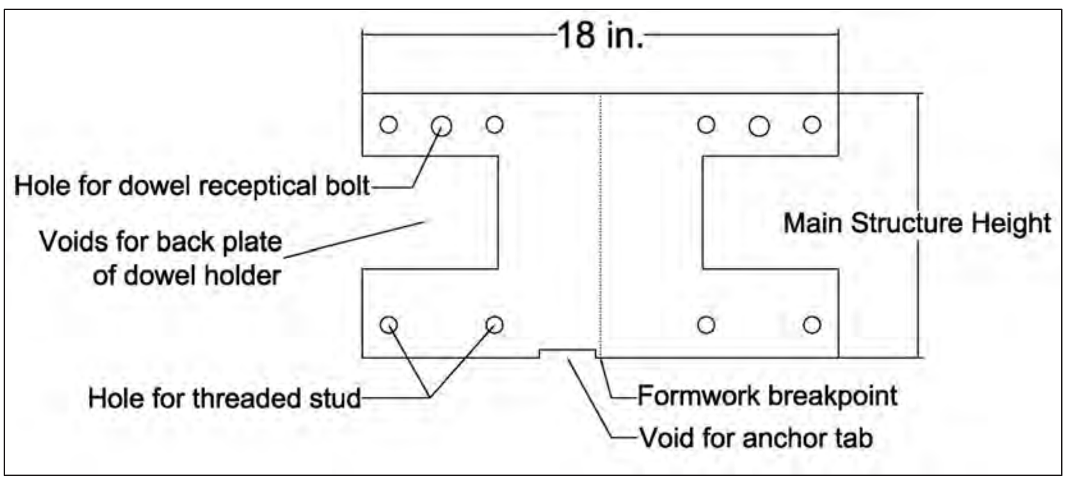

Figure 29. Connection plate concept drawing.

arrangements such that one plate design was applicable for accessory items placed on the back of the formwork. The large horizontal cutouts allowed for installation around the back plate of the dowel holder assembly. A smaller vertical cutout was needed at the form depth for placement over the anchor tab. The larger diameter holes at the top of the plate allowed for the dowel receptacle bolt to be placed.

Structural design verification

Precast concrete panels must be designed with adequate steel reinforcement to prevent damage from loadings that induce significant tensile stresses. Figure 30 shows the areas of concern for a typical precast panel where overloading may occur. ACI 318 requires precast concrete be designed for multiple loading cases since higher stresses may be developed between casting and final installation rather than during service. This is especially true for precast concrete pavement since the support conditions of a slab on grade (elastic foundation) are significantly different than structurally supported concrete. The reinforcement from the original design was analyzed using the estimated applied loading to ensure enough reinforcement was provided. Additional material design considerations were selected to assure panel performance and quality.

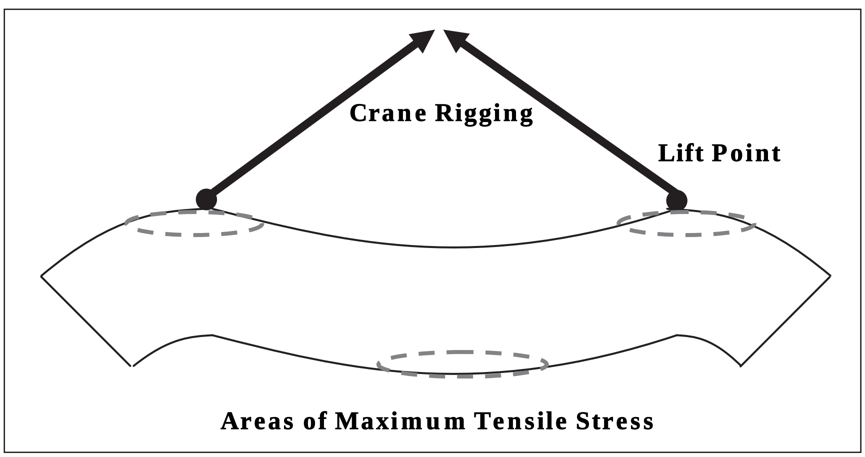

Figure 30. Areas of maximum tensile stress when a structural element.

Material selection

Steel reinforcement

The original Air Force panel had two different sets of reinforcement. The main structural grid was placed towards the depth of the slab and consisted of two layers of #3 (0.375-in. nominal diameter) bars at 12-in. spacing on center. Nine bars were used in each perpendicularly placed layer. A layer of #5 (0.625-in. nominal diameter) was placed on top of this grid to provide additional flexural support. An additional grouping of #5 bars was placed above the mid-height to stiffen the doweled area and decrease any panel deflection when loaded as a pavement. The reinforcement locations differed between the panel types because of the precasting of the dowel receptacles. No reinforcement was specifically provided to support the tensile stresses generated from the cantilevered area between the lifting points and the slab edge; however, the #5 bar placed above the mid-height was capable of providing this support.

Grade 60 (60 kip/in^2 minimum yield strength) reinforcement was selected because it is commonly used and widely available. Since design calculations were completed using this material, only grade 60 bar or greater can be used for construction. The panels cast at the ERDC used ASTM A615 steel placed in the configurations shown in Figures 31 and 32; however, other ASTM compositions may be acceptable. Plain steel is expected to be used for construction since it is commonly available, but ASTM C775 epoxy coated reinforcement can also be used if corrosion is a concern.

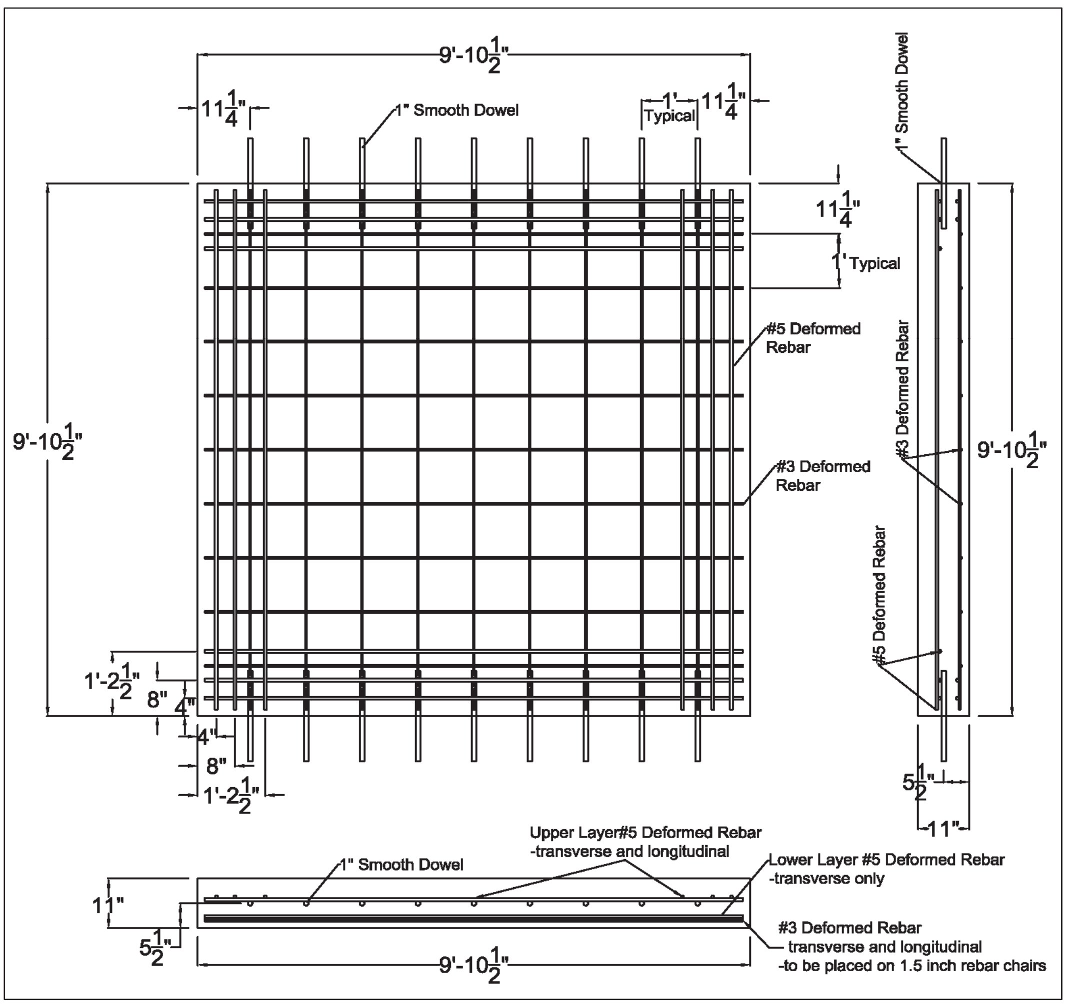

Figure 31. Rebar layout for standard precast panel.

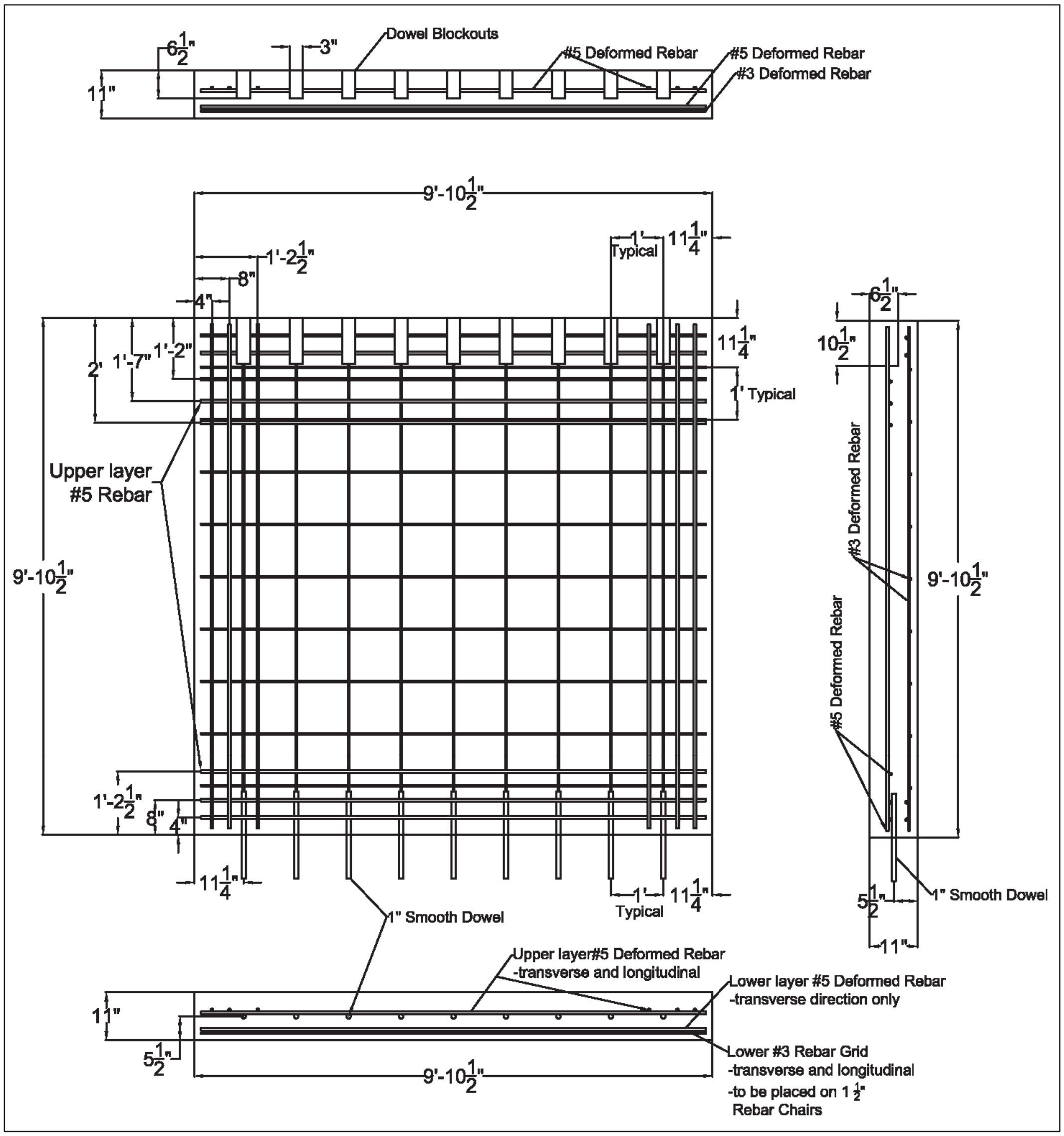

Figure 32. Rebar layout for terminal precast panel.

Concrete mixture design and materials

The concrete material used for panel construction was important for both the pavement and structural design of the precast panels. Review of the original design showed no requirements presented for the concrete mixture past the compressive strength needed. A list of requirements, along with typical testing procedure standards, was developed to assist with ordering quality material, aid in construction, and ensure panel long-term durability. Use of the exact mixture design used for any previous field testing is not required since proportioning of concrete mixtures with local materials will differ.

A concrete with a 5,000 lb/in.^{2} design compressive strength at 28 days was selected for use as it is a commonly available material from ready mixed concrete suppliers and was maintained from the original design. Mixtures of this strength will be comparable to a 650-lb/in.^{2} flexural strength mixture typically used for cast-in-place airfield paving construction. Mixtures with similar mechanical properties are required to maintain the structural design of the panels.

The following concrete mixture properties are minimum requirements when constructing panels to comply with structural design calculations, ensure their proper construction, and mitigate environmental damage when stockpiled. The items detailed below were adopted from Unified Facilities Guide Specifications (UFGS) 32.13.11: *Concrete Pavement for Airfields and Other Heavy-Duty Pavements*. Typical ASTM International testing procedures were also provided to assist with material acquisition planning and construction monitoring. Use of excessively stringent requirements may make implementation of the precast panel system difficult for contingency environments due to their locations or materials present.

- The PCC mixture must have a minimum 5,000 lb/in.^{2} compressive strength at 28 days in accordance with ASTM C39 to comply with the structural design for lifting and storing the panels. Formwork may only be removed after the concrete gains 2,500 lb/in.^{2} or after 7 days of curing to prevent damaging the panels.
- The maximum aggregate size shall be no more than 1 in. in accordance with ASTM C136, and any gradation used should follow the limitations given in ASTM C33. This will ensure the mix can be consolidated around the reinforcement.
- Aggregates used should be frost resistant to prevent durability cracking in the panels when exposed to wet-dry and freeze-thaw conditions. The 1-in. maximum aggregate size used will assist in achieving this criterion in many instances.
- An air content of 6±1 percent in accordance with ASTM C231 or C173 is required to prevent freeze-thaw damage to cementitious paste from outdoor storage in cold-wet climates with repetitive freezing and thawing.
- The concrete's workability (slump) should be 4 ± 1 in. in accordance with ASTM C143 to allow for efficient placement in the forms, around any receptacles formed and for effectively embedding any steel.

Reinforcement cover

Depth

The original design only provided 1 in. of PCC cover depth. The reinforcement cover depth was updated to follow ACI 318 guidance for cast in place concrete to protect the reinforcement from corrosion. The minimum cover depth was selected to be 1.5 in. between the reinforcement grid and the bottom of the panel (chair height) for concrete exposed to earth or weather. A spacing of 1.5 in. between the reinforcement ends and the formwork was also specified.

Accessories

Steel chairs that provided the correct cover depth were specified for supporting the reinforcement at the correct location. Specifying steel ensured wooden materials were not used. Commercially sold ceramic, thin plastic or concrete mortar items may be acceptable if good bond is provided to the concrete. Chair heights should be verified to ensure the bottom of the bar is placed at the cover height and not the bar mid-height. Chairs that use minimal material and allow concrete to flow under and around the supporting chair were highly recommended to ensure the chaired area becomes an integral part of the panel. Standard steel wire bar chairs were not expected to exhibit any bond or concrete embedding problems. Base plates can be ordered for wire chairs if placement on granular surfaces is necessary.

Standard 16 gauge low carbon steel wire was used to fix bars to the chairs and bars to each other at their intersections. Spooled wire cut on site or precut ties were both considered acceptable. Precut ties should be long enough to completely encircle tied items and produce a tight, rigid connection. The panels constructed by the ERDC used 6-in.-long precut ties at #3 bar intersections and for tying to chairs. A longer 8-in. precut tie was used for typing #5 bars locations.

Load demand and structural capacity determination

The applied loading, support conditions, and geometry of the structure affect the response of a structure. To accurately determine if enough structural resistance was provided to prevent damaging the panel before installation, the panel's structural capacity was determined and compared against the demands of the loading generated. Since lifting and storage conditions may generate different support conditions, separate analyses of the scenarios were required.

Design loads

Load factors and combinations were applied following American Society of Civil Engineers (ASCE) 7-05. The self (dead) weight load combination was the only combination to apply to the lifting and storage situations because there were no live loadings with large accelerations or significant environmental loads expected. Dead loads increased by 40 percent (load combination 1 by Load and Resistance Factor Design, LRFD) of their expected value for this situation. The panel may experience small acceleration dynamic loads during transport and handling and are difficult to estimate. These loads are typically accounted for by increasing the static load to compensate (Wight and MacGregor 2009). No guidance for selecting a value to increase loads by was found in literature; therefore, a value of 50 percent was selected and assumed reasonable. The worst case scenario of the two options described above was applied and used for the design load. The unit weight of reinforced concrete applied for the dead load was 150 lb/ft^3; therefore, the resulting design load was 225 lb/ft^3.

Lifting

The panels were designed as a simple beam with two support reactions located at the lifting points. The panels were designed to be square; therefore, the loading was the same in both directions. The original design used anchors inserted 3 ft from the edge; however, this location was later modified to 2.5 ft to minimize the resulting stresses across the length of the lifted panel. Moving the lifting point closer to the formwork also assisted with field construction. Placing the rigging hardware in the designed locations is crucial to ensure the panel reacts as designed. Figure 33 shows the bending moment generated by the design load across the length of a panel. The various lines plotted show the effect of lifting point location on the bending moment generated.

The precast panels were designed to be lifted without cracking. The panels were analyzed to verify their un-cracked nominal moment strength. To prevent the smallest crack from initiating, the largest tensile bending stress was limited to a value less than the tensile strength of the concrete. The American Concrete Institute (ACI) recommends using the modulus of rupture (flexural strength tested by ASTM C78) as the tensile strength

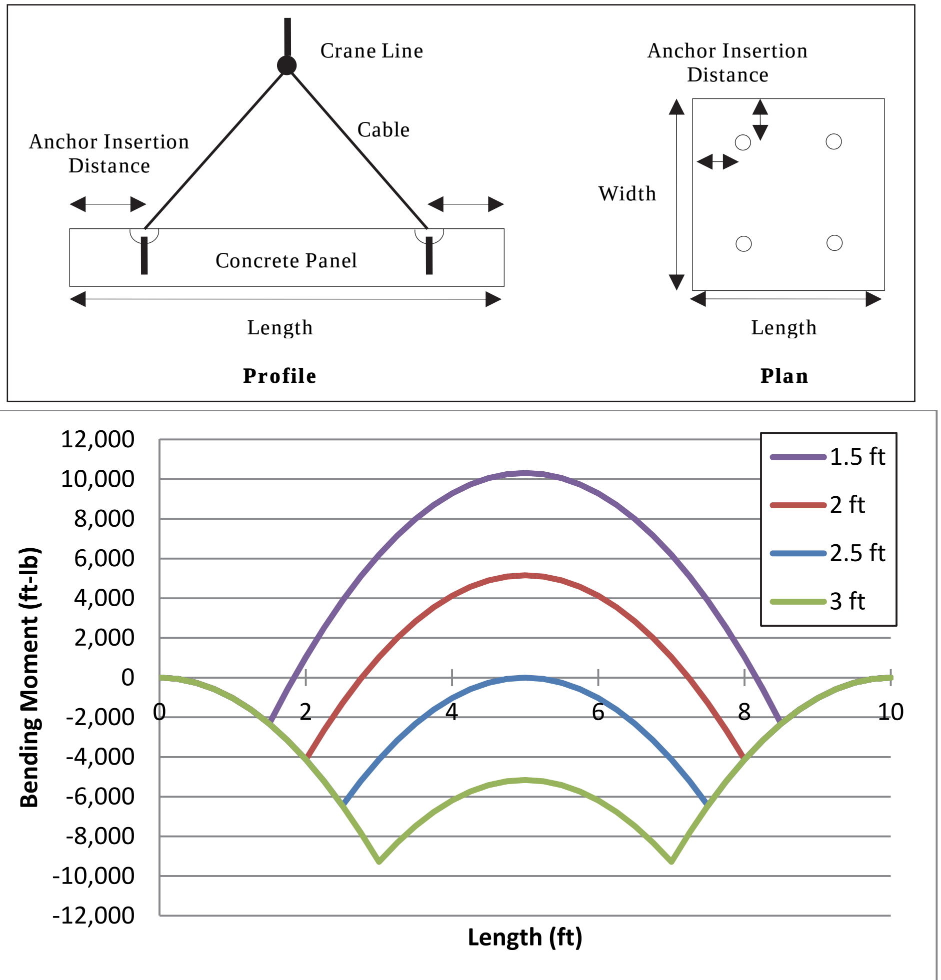

Figure 33. Comparison of bending moment generated by different equidistant anchor insertion locations.

limit and applying this value to the standard flexure stress equation. Equation 5 details the flexural stress equation used to find the maximum bending moment to prevent cracking with ACI recommended values. This equation uses the gross cross-sectional moment of inertia that considers the concrete only and not the effect of any reinforcing steel. Inclusion of the steel will produce negligible differences in the values. Also since crack initiation will be begin at the surface (extreme tensile fiber), the distance from the neutral axis is half the panel height. The values resulting from the analysis were the same for both the longitudinal and transverse direction because the panel dimensions in both directions were the same.

$$\sigma = \frac{Mc}{I} \rightarrow M_n = \frac{\sigma_{f_r} I_g}{c} = \frac{\left(7.5\sqrt{f'c}\right)\left(\frac{1}{12}bh^3\right)}{\frac{h}{2}} \tag{5}$$

where

M_n = Maximum bending moment to prevent cracking
σ_{fr} = Modulus of rupture
I_g = Gross moment of inertia
c = Distance from neutral axis to a specific point cross-section (half the panel height for investigating tension at panel surface).

Table 6 shows the nominal maximum bending stresses allowed to prevent cracking of the concrete. Values for different compressive strengths were reported to provide insight for lifting restrictions for the selection of rigging hardware components. The calculated design moment strengths shown were reduced with a strength reduction factor (ϕ) of 90 percent for tension controlled sections. Comparison of the values against the plot provided in Figure 33 shows that concrete alone provides significant tensile capacity to prevent cracking when lifted for each of the lifting point locations considered. This is expected since the panel's self weight is relatively light compared to the panel's relatively large cross-sectional area that provides significant resistance.

Table 6. Maximum bending moment allowed for various concrete strengths.

Minimum concrete compressive strength when lifted, lb/in.^{2}	Design aspect considered	φM_n, ft-lb
5,000	28 day design strength	96,250
3,500	For shortest 4-ton lifting anchor available	80,530
1,800	For anchor specified	57,750
1,600	For longest 4-ton lifting anchor available	54,450

Storage

For space considerations, panels were allowed to be stacked when stored (Figure 34). Panels are typically stored on two pieces of dimensional lumber dunnage placed parallel to the dowel bars. Pressure treated lumber with dimensions of 4 in. x 4 in. x 10 ft was selected for dunnage material since it is relatively easy to handle with two people, spans the length of a panel, allows for air movement and water drainage between separate panels, and resists decay over time. Designing with additional dunnage past two pieces is not recommended because damage may result from differential settlement of the supports.

Figure 34. Panels stacked on dunnage.

The dunnage locations and orientations are extremely important because the storage stresses typically govern the precast panel design. When panels are stacked on supports that are not in their intended design locations, the misalignment applies unintentional additional load to the misaligned panel and any panels stacked below. The greatest concern is damaging the bottom panel due to the total weight of all the panels stacked above.

Since the weight of the panels above the lowest panel in a stack ultimately affects the loading, the maximum number of panels in a stack was set to limit the reinforcement required. The stacking height was limited to less than 6 ft to eliminate the need for fall protection under Occupational Safety and Health Administration (OSHA) regulations when installing the crane rigging. Four panels plus the height of four pieces of dunnage meets this requirement with a combined height of 5 ft.

The panels were designed for stacking with dunnage placed beneath the lifting points to provide an obvious reference for personnel to use. The loading applied to the lowest panel will be similar to that of the dead load when lifting; however, two additional point loads are also applied resulting from the weight of stacked panels above when dunnage is misaligned. The point loads were applied equidistant from the ends. Placing the point loads directly above the supports does not generate additional loading in the lowest panel. However, moving the point loads equally to the right and left of the supports generates additional load on the lowest panel.

Figure 35 shows the additional loading created from misaligning the dunnage above the lower most panel (support dunnage held constant below lifting points, part a) and the lower most panel's dunnage itself (stacked panel dunnage held constant above lifting points, part b). The various lines show the bending moment generated for the first panel stacked as a result of misalignment.

The modified precast panel's reinforcement design was analyzed to determine the allowable tolerance the dunnage could be misaligned without overloading the lowest panel. Table 7 details the calculated nominal moment strength of the panel for the different cross-section and tension zone locations. Both cross sections were treated as singly reinforced sections using typical reinforced concrete design methodology. The #3 and #5 bar development length for plain steel rebar was verified, and both sized bars are fully developed before reaching the vicinity of the lifting point area (8 and 14 in. minimum length, respectively). All cross sections considered are tension controlled; therefore, the strength reduction factor required was 0.9.

Comparing the values in Figure 35 against those in Table 7 shows the lower portion of, the worse case loading considered was misaligning the lowest panel's dunnage. Calculations showed the lower portion of the longitudinal cross-section limits the misalignment allowed. The capacity of this cross section was reached with dunnage placed approximately 11 in. to the right from its intended location. The allowable dunnage placement tolerance was set to ±9 in. of its design location for constructability. To achieve a nominal moment capacity greater than that required for a 12 in. misalignment of the lowest panel's dunnage, adding an additional #3 bar to the lower layer of the grid was required. This addition was recommended to provide an easy, constructible value personnel would be able to measure easily.

Table 7 also shows that the amount of reinforcement in the other cross-sections for structural proposes can be decreased for more economic panels since all other capacities shown are at least 33 percent greater than required at the ±9 in. dunnage tolerance selected. Using smaller #4 bars in the upper cross-section locations and removal of the #5 bars from the

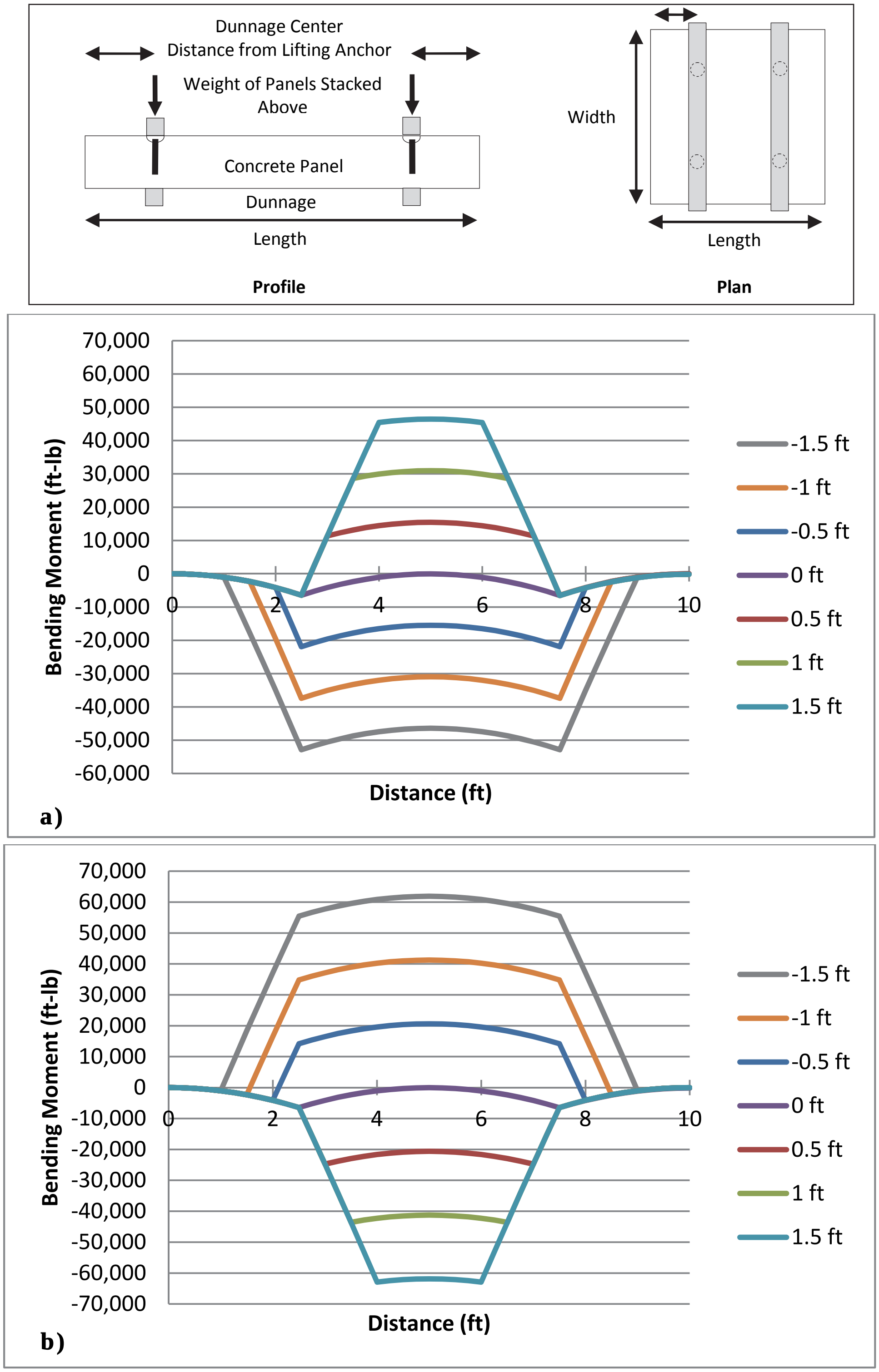

Figure 35. Comparison of bending moment demand by misaligned dunnage. a) dunnage for panels stacked above the lowest panel and b) dunnage placed for the lowest panel.

Table 7. Nominal moment strength provided by reinforcement configuration.

Direction	Location in cross-section	Steel area (A_s), in.^{2}	Effective depth (d), in.	Nominal Moment Capacity (ΦM_n), lb-ft
Longitudinal	Upper	1.86	6.94	- 57,100
	Lower	0.99	8.94	+ 39,500
		1.10	8.94	+ 43,900
Transverse	Upper	1.86	6.31	- 51,800
	Lower	2.23	8.83	+ 87,100
			8.48	+ 82,600

lower portion of the transverse cross-section may produce an optimized design as a cursory estimate; however, any optimized reinforcement configuration will require analysis to verify its adequacy.

Future reinforcement considerations were also made to allow the panels to be stacked in any direction to remove the risk of damaging panels when stockpiled incorrectly. Placement of additional #5 reinforcement perpendicular to that currently provided in the upper portion of the panel was required to support dunnage perpendicular to the dowels.

Table 7 documents the analysis conducted by adding the additional reinforcement to the longitudinal direction in the upper cross-section of the slab. Reinforcement was expected to be tied on top of the currently specified #5 bars placed above the dowel bars at similar distances away from the formwork face. This additional reinforcement was more than sufficient to support the stockpile loading considered.

Transport demand

Transport demands were not an issue since panels were not expected to be delivered stacked. Available truck and trailer load capacities were not expected to allow delivery of more than one or two panels at a time. Therefore, moment capacity and demand were similar to the lifting cases where the panels are not cracked.

Additional ACI reinforcement recommendations

ACI recommendations for reinforcement spacing and the inclusion of minimum reinforcement steel for slabs were not applied on the original reinforcement arrangement. Future modifications to the reinforcement layout should address these items for both the longitudinal and transverse directions for stricter adherence to reinforced concrete design standards. Discrepancies include:

- Achieving the 18-in. minimum reinforcement spacing for panels may be difficult with larger bars. Use of a larger quantity of smaller bars spread out across the length or width will help meet this requirement.
- The minimal amount of steel required by ACI 318 for each of the cross-section combinations considered in Table 7 is 2.38 in.^{2}. All cases considered require additional reinforcement.
- ACI 318 requires that parallel reinforcement placed in multiple layers have a clear spacing of at least 1 in. or the nominal bar diameter if larger to allow concrete to easily flow between the layered bars during construction and prevent concentrating reinforcement. Only the transverse cross-section does not meet this criteria since the #5 rebar tied to the grid is only separated from the parallel reinforcement by 0.375 in. Placing the #5 rebar currently tied to the reinforcement grid on a 3-in. tall reinforcement chair would assist with meeting this criterion. The nominal moment strength computed for this design of the transverse lower reinforcement results in a minor 6 percent reduction in this cross-section's nominal moment strength; however, much more strength is present than needed for stacking loads. This is only required if the cross section is not optimized, and the #5 rebar from the grid is not removed.

Bridge plate design

Bridge plates were designed to suspend the precast panels over the bedding material until it gained sufficient strength to support the panel. The original design attached the bridge plates to the panel before lowering into the pavement void (Figure 36). The superstructure support design was continued from the original design because it provided an easy visual indication that the surface of the panel was flush with the existing pavement after installation. Adjustments could be made by using different thickness wooden shims stacked under the bridge plate.

Structural considerations

A steel channel section was selected for use to allow the bridge plates to mount flush against the pavement after installation, and the flanges provided the additional structural resistance to suspend the panel over the

Figure 36. Bridge plate installation.

flowable fill. Selection of a minimum channel section followed a similar process as that shown earlier for the formwork, except that different loading cases and support conditions were considered to determine the minimum size needed. The model considered used a cantilevered beam with a point load applied at some distance away from the support (Figure 37). Equation 6 modeled the situation. The model assumed the flowable fill offered no support, and the bridge plate was fixed to either the parent or adjacent pavement when installed. The point load applied was located in the area between the three holes in the channel's web where it connects to the panel's anchoring. The load location was assumed to be closer to the two holes towards the construction joint since more anchors were placed there. The point load location used for calculations was placed at 4 1/6 in. from the construction joint based on weighting hole locations from their distance from the joint.

$$\Delta_{\max @ x} = \frac{Px^3}{3EI} \rightarrow I_{\min} = \frac{Px^3}{3E\Delta_{\max}} \tag{6}$$

where

E = Material modulus of elasticity, 29 x 10^6 lb/in.^{2} for steel
I = Cross section moment of inertia, in.^{4}
x = Point load location, in.
Δ_{max} = Maximum allowable deflection, in.
P = Load, lb.

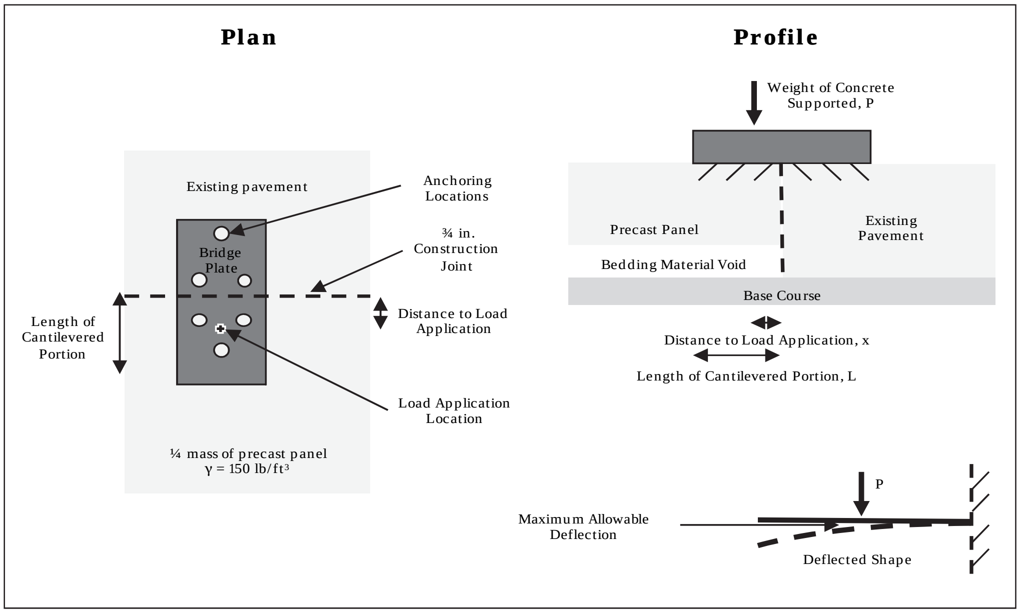

Figure 37. Free body diagram of loading on bridge plate.

Table 8 shows the moment of inertia needed for the bridge plate at different stages of panel installation. The original Air Force system utilized a smooth drum vibratory soil compactor to assist in panel seating if the panel could not be placed flush with the existing pavement. Adding the additional weight and operation of the roller added significant loading to the bridge plates. Modeling the static roller was difficult because equipment inventories may vary. The research team believed a CAT CS-433E vibratory roller was an appropriate machine available to Air Force installations. This machine has a gross vehicle weight of 15,230 lbs (with leveling blade) and a drum weight of 8,615 lb (with leveling blade). A loading of 9,000 lbs and 18,000 lb was assumed for the additional live load with the roller alone and when vibrating, respectively. Surface flushness was maintained by selecting a very small allowable deflection. A value of 1/250 in. was assumed reasonable and limited the need to select an excessively stiff and heavy channel section.

The design loading described earlier could not be applied to this situation since the roller added a live load on the system. Revisiting the LRFD load combinations in ASCE 7-05, the appropriate total design loading for the added live load was where the dead load is increased by 20 percent, and the live load is increased by 60 percent of their expected values (load combination 2). The minimum moment of inertia required was 3.3 in.^{4} for steel bridge plates. The minimum channel section that meets this requirement is a C10 x 25; however, a C12 x 25 was locally available and used for field testing.

Table 8. Determination of minimum bridge plate size.

Load description	Component loads on panel, lb	Number of supporting bridge plate	ASCE 7 LRFD load combination used	LRFD factored load on individual bridge plate, lb	Minimum moment of inertia, in.^{4}
Panel weight only	Self weight - 13,750	4	1	4,813	1.0
roller slowly moving into position	Self weight - 13,750 Live load- 9,000	2	2	15,450	3.3
Roller operating with vibratory pulses at center	Self weight - 13,750 Live load- 18,000	4	2	11,325	2.4

Connection modification

The original design attached the bridge plates to the panels before installation. The connection was made with wet-setting high-strength threaded inserts embedded in the wet PCC during panel construction (Figure 38). The inserts accepted 0.75-in.-diameter bolts with washers to complete the connection to the bridge plate. An additional structural (square) washer was used to further stiffen the connection. Steel plugs were used to protect the threaded interior portion of the insert from concrete during panel construction and corrosion when stockpiled.

Figure 38. Wet-set bridge plate connection inserts.

Preliminary field testing identified issues with the threaded inserts. The manufacturer called for a minimum edge distance of approximately 6.5 in. When the panels were suspended with the inserts installed at the minimum edge distance, a 0.25-in. separation between the top of the panel and the bottom of the bridge plate was observed. This was attributed to the edge distance of the installed threaded inserts (load application distance on the cantilevered portion of the plate), as there was no significant deflections evident in the bridge plates when they were examined after testing.

A second issue was encountered when installing multiple panels such as a quad-panel repair. Use of this configuration removed the attachment points at the interior intersection of the panels and required rearrangement of the bridge plate locations. A diagonal bridge plate was added to the equipment package to allow support at three corners (Figure 39); however, attempts to leave the interior panel intersection unsupported were unsuccessful. The unsupported corner caused the inserts to pull out from the slab. The research team believed the simplest option was to provide support to this portion of the panel with a small concrete disk placed on grade before placing the bedding material (Figure 39). A concrete disk 2-ft in diameter was cast during panel fabrication at the ERDC for the test section. The height of the disk was equal to the difference between the existing pavement and panel thickness (3 in. for the ERDC test section). The future disk design was modified after test section construction to include smaller, individual 1-ft diameter disks placed under each panel's interior corner to reduce its overall weight.

Figure 39. Concrete support disk and diagonal bridge plate for quad repairs.

Consequently, a new bridge plate attachment anchor was identified and subjected to field testing. A 0.5-in. wedge-type anchor, shown in Figure 40, met the required tensile load capacity and allowed for installation as close to the edge of the panel. The anchor model used for this task was previously tested by the ERDC for repair matting pavement anchoring with good performance.

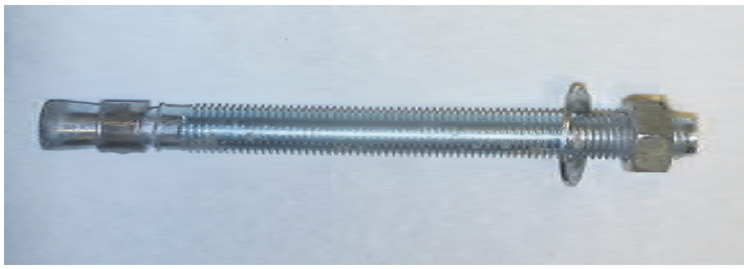

Figure 40. Concrete wedge anchor.

The wedge anchors also aided installation efforts by providing flexibility and greater accuracy in positioning the bridge plate on the panel or existing pavement. The threaded insert locations were fixed after casting and did not allow major positioning modification. Fabrication errors or damaged anchoring would render panels useless since the bridge plate connection points could not be modified. Installation of the wedge anchors required minimal equipment additions, and anchor holes were made very quickly in the field as needed.

Lifting equipment considerations

The system required rigging to attach the panel to the crane. Adequate rigging hardware was specified to prevent injury to personnel, damage to the precast panels, efficient removal of existing PCC, and for ease of use.

Equipment setup

The rigging system package was designed as a four-leg bridle configuration where four separate cables were used to connect the lifting points to the crane line. Even though no load was carried in two of the legs of the four-leg bridle system when rigid panels were lifted, greater load stability was achieved when compared to a two- or three-legged system. All lifting points were placed equidistance from the panel corners to allow for equal loading on all components and for centering of the center of gravity below the crane line for stability.

When verifying and selecting components for the rigging package, a minimum 4:1 safety factor was maintained following standard industry practice for reusable commercial pieces to prevent damage from repetitive use. Additionally, vertical sling hitches were used when selecting the load capacity of the slings. Use of reeving, where a single sling is used over multiple connection points, was not recommended to prevent pinching and wear of slings and overloading of the connection equipment.

Rigging package components

Proper connection of the precast panel to the crane required selecting the various components of the rigging system for the lifting job completed. Optimizing the components to the expected loadings balanced the equipment needed for safe and efficient lifting of the precast panels. Some components described were not directly specified in the original design. Information on the original equipment types needed was scarce and estimated from photos (Figure 41). Once typical equipment required to complete the lifting was determined, rigging was selected based on the expected design loadings.

Figure 41. Original rigging design photos.

Sling

A sling is a cable that attaches the hook of the crane to the lifting points embedded within the precast panels. The angle the sling makes with the panel ultimately determines the loading it experiences during a lift. Selection of the appropriate rigging geometry depends on many factors including the crane used, load stability, and selection and economy of other components selected. Generally, selecting shorter slings or placing the lifting points closer to the edge of the panels decreases the resulting sling angle. This in turn increases the amount of shear force on the system and results in selecting more expensive, heavier, higher capacity slings from the additional total load added.

The sling angle was determined by the length of the sling and the positioning of the lifting point the sling connects to, as shown in Figure 42. Selection of a commercially available sling involved selecting the first available product that meets or exceeds the value determined for the hitch configuration used. Typical posted ratings for commercial equipment have a safety factor built into the value shown, and the value determined for the sling load did not need to be multiplied by the safety factor. For lifting the modified precast panel, the system required a sling rated for 3.83 tons or greater for a vertical hitch using an 8 ft sling.

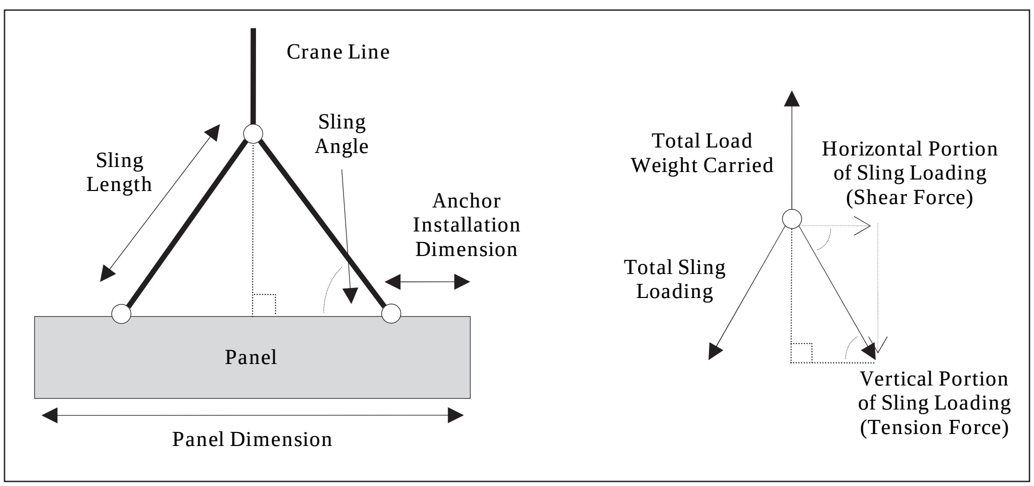

Figure 42. Geometry and loading diagrams of sling rigging.

The equipment package used for this task was designed to lift up to an 18-in. thick slab. To prevent purchasing additional equipment, the sling selected was designed to work for lifting a new precast panel and removing existing monolithic sections of concrete. Since the maximum thickness of the damaged slabs increased the load the sling would experience compared to a precast panel, a higher rated sling was required. An 18-in. slab approximately 10-ft-square weighs about 11.3 tons. Using the 30 in. anchor installation distance from the slab edge requires a minimum sling capacity of 6.3 tons with a vertical hitch using an 8 ft sling.

Initially, 16 ft sling lengths were selected to help reduce shear forces generated on the damaged pavement removal anchors used (discussed later in section). This was later decreased to 8 ft after experiencing difficulty installing precast panels at the ERDC test section using a 22-ton crane. This crane capacity was closer to the specified minimum capacity than the 50-ton capacity crane used for the initial 16 ft sling testing. The longer slings required a longer boom radius to lift slabs, which in turn decreased the crane's lifting capacity. Since lighter cranes were expected in the field, the lifting capacity for longer slings may not be available. Use of slings longer than 8 ft will require verification against the selected crane's load chart before use.

The type of sling used was based on its ability to be user friendly. The design team selected a synthetic material that would be easy for personnel to move and would not cut skin like abraded wire ropes. A synthetic, endless round sling was selected because it was easy to use, economical, and lighter weight than steel. The load bearing strands were protected from abrasion, ultraviolet light and decay by a non-load bearing synthetic jacket. The synthetic sling also offered corrosion resistance from long term kit storage.

Shackle

Shackles are used to connect a sling to the equipment lifting point. Shackles that can be quickly taken apart, but securely put back together were essential to installation speed and safety. Based on the required sling capacity for removing 18 in. slabs, a 6.3-ton minimum working limit shackle was required. A bolt type shackle was selected as opposed to a screw pin shackle, based on its ease of use and safety benefits. A model with a locking pin was chosen to prevent the nut from loosening and to ensure the bolt stayed in place during the lift. The stock unit specified came with a cotter pin as the locking pin; however, this was replaced with a quick releasing safety pin for faster and easier operation.

Panel lifting anchor system

An anchoring device was installed into the cast panels to provide a rigging attachment point. The Dayton Superior Swift Lift® system was selected to allow for ease of installation, expedient connection of the panel to the crane line, and disengagement prevention system.

The lift system consisted of a steel anchor (part P-52) embedded in the concrete and a lifting eye attached to the anchor and the crane rigging (shackle). A hemispherical void (part P-56-PL) was formed at the top of the anchor when the panel was cast by a removable rubber plug. The void allowed the system to rotate as needed during lifting to minimize restraint. Since the panels will not be rotated vertically after casting, a basic eye, (part P-51), was deemed appropriate.

The selection of the lifting anchor components depended upon the sling load experienced. Since both items will only be used on the precast panels, the minimum sling load of 4.0 tons was used for the minimum capacity.

The selection of the embedded anchors required balancing the strength of the concrete when lifted against the positioning of the anchors from the edge and other anchors. The edge distance was preselected by placing the lifting points at 30 in. from each edge. Additionally, the minimum spacing between anchors required was twice the anchor's allowable edge distance. The minimum edge distance allowed for the 4-ton anchors was 20 in., resulting in a 40-in. spacing between anchors. The clear spans between supports were 60 in. This was larger than the 40-in. minimum spacing and provided adequate distance to be used. Activation strength of the concrete was not a significant design factor for anchor selection because the panels must gain 5,000 lb/in.^{2} compressive strength before moving. This was well over the 3,500 lb/in.^{2} minimum strength required to use any 4-ton anchor available. Considering the anchors available and the loading points used, the minimum anchor length required is 5.75 in. for the model chosen. The 7.125-in. model was selected since its true load capacity was equal to the nominal capacity needed.

Damaged pavement removal anchor system

One of the most time-consuming tasks during the installation of a precast panel is the removal of the damaged in-place pavement. Traditionally, damaged pavement was demolished using a hammer attachment to an excavator or skid steer and removed using front end loaders or skid steers with bucket attachments. Current precast paving operations described in the literature removed existing pavement in large, intact units. The Air Force method used wet-setting anchors.

Figure 43 details the original wet-set anchor installation method used for pavement removal. After cutting the perimeter of the pavement to be removed, a 6-in. diameter core was removed from each lifting point. Rapid-setting rigid pavement repair material was mixed and placed into the core hole to embed a Dayton Superior Swift Lift® anchor into the pavement using Pavement 15®. The anchor and lifting eye for this procedure required selecting 8-ton nominal capacity anchors to allow for removing pavement thicker than the 11-in. panels. After embedding the anchor, the repair material was allowed to cure to the anchor's activation strength before removal of the damaged portion of the slab (Ashtiani et al. 2010).

Figure 43. Wet-set anchor installation.

While this procedure was fairly simple and cost effective to implement, two issues were observed with the repair material used. The rapid-setting repair material required a significant amount of time to cure to the required anchor activation strength. Materials certified for small patches require compressive strengths of 3,000 and 5,000 lb/in.^{2} in 2 and 24 hr, respectively. Possible anchor candidates would require activation compression strengths of at least 3,500 lb/in.^{2}, resulting in more than 2 hr of time spent on this task alone.

Repair times would be further extended if the repair material failed to bond to the smooth interior surface of the cored hole and additional attempts at embedding an anchor were required.

A more rapidly installed and reliable lifting anchor was required to meet the repair times desired by the Air Force (4-8 hr). If rapid-setting repair material was required for this task, a significant volume would be required in the specified supply package.

An alternative to rapid-setting concrete repair materials was the use of a mechanical concrete anchor. Typical products involve driving a pin or a bolt through a mechanism that expands and attaches itself to the concrete. Review of commercial products showed that Simpson's Torq-Cut® (Figure 44) undercutting anchor was the only product available that could support the removal capacities required for 10 to 18 in. thick slabs. Equipment required to install the Simpson anchor included a dry cutting masonry drill bit, a drill compatible with creating large holes in concrete, and a compressed air source to clean the hole after cut. All this equipment was significantly less expensive and easier to operate than coring equipment and did not require a water source. Most importantly in terms of the anchors operational effectiveness, the anchor can be used immediately after inserted and tightened to its installation torque.

Figure 44. Concrete expansion anchor equipment.

Use of the mechanical concrete anchor required selecting additional rigging hardware to connect the threaded top section of the anchor to the shackle. A swivel lifting point with a 6.3-ton minimum capacity was selected to provide the attachment point. The stock lifting hoist was modified to accommodate the expansion anchor by removing the bolt at the center of the unit. A simple, custom steel bushing was machined to fill the additional volume remaining after the expansion anchor's threaded rod was placed through the hoist ring hole. The bushing provided a surface for the expansion anchor nut and washer to bear against for anchor activation.

For the Simpson mechanical anchor, its strength was not specified unlike the Dayton Superior wet set anchor. Dayton Superior anchors are listed by their nominal capacity in any direction, implying that the combined shear and tension loading applied by the rigging have been factored into the value given. The Simpson anchor capacities for combined loadings required calculation to determine the safety factor available. Calculations comparing the shear and tension forces generated by lifting slabs with different thicknesses to manufacturer anchor strengths requires the factor of safety to range from 3.0 to 1.6 for 10 to 18 in. thick slabs, respectively. Reductions in the safety factor deviate from the criteria used for other rigging components (safety factor of 4). This reduced value was considered acceptable since the anchors are loaded well short of their ultimate strength and the anchors are disposable.

Return to service requirements

To ensure the bedding material was strong enough to support the completed panels during placement, an investigation into the flowable fill material requirements was conducted. The investigation focused on determining minimum strength requirements needed before trafficking and developing guidance on material selection to best allow users of the information to tailor the material to local availability and situational needs. Use of this information allows for efficient planning and preparation for the work tasks, yielding speedy repairs.

Minimum bearing capacity

The bearing capacity of the underlying soil beneath PCC pavements is an important design consideration where the amount and uniformity of support dictates a pavement's performance. Generally, rigid pavements are more forgiving to soil that offers little support since applied loads are spread across the slab area. However, the precast panel system is different from typical construction because hardened panels are placed on fresh flowable fill bedding that is gaining strength over time. Because of this, the repair may not be ready for traffic immediately after installation.

To encourage good performance of the completed repair and to minimize additional repair effort, the minimum strength required to load flowable fill without damage was determined to keep plastic flowable fill from squeezing out from construction joints and sink into the filled void or crush under load. Having an accurate estimate of the strength required also provided an accurate estimate the overall repair timing needed.

Modeling the panel as a shallow rectangular (spread) foundation with a rough base was considered to be the best approach to determine minimum bedding material strength needs. Both the modeled and actual situations are similar in theory, where a rigid unit punches through a softer base. In this situation, the precast panel is forced down by an aircraft load. Rotation of the panel is not expected since the existing pavement bounds the repair to limit horizontal movement and plastic flowable fill will be significantly weak across the panel dimensions.

Deviations from the model included the dowels used and the dynamic load aircraft applied when utilizing the pavement. First, the dowels embedded into the existing and adjacent pavement will take on some of the applied load. However, expecting the dowels to function properly may not be reliable since the rapid-setting repair material used to grout the dowels into place requires time to gain strength as well. The repair material may not rigidly embed the dowels into the system when trafficking begins. Assuming the material is potentially plastic maintained the foundation theory, and the panel could settle 0.5 in. before bearing on the concrete at the receptacle depth. Second, the foundation model assumes static loads and not the dynamic loading the aircraft supply. The loading model simulates an aircraft parked directly over the repair. Moving aircraft are expected to place less load on the pavement than stationary aircraft, and the panel will not require the bearing capacities calculated.

The required bearing capacity depended on the amount and positioning of the overall load applied. As the panel load moved away from the center of the panel, the loading became eccentric, and the amount of panel area bearing on soil decreased. Since the load was distributed over a smaller area, the stress on the underlying soil increased and would cause failure if the underlying soil was not able to resist the load. Figure 45 shows the previously described situation.

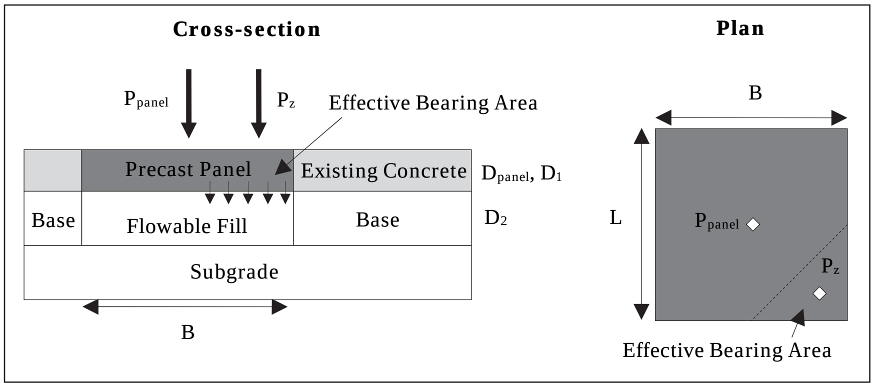

Figure 45. Schematic of bearing capacity modeling.

To determine the bearing capacity needed for aircraft traffic, the axle configurations of two different typical aircraft were considered. Figure 46 and Figure 47 detail the critical load configurations the precast panels may experience under C-17 and F-15E aircraft, respectively. Each case either contained gear combinations that maximized the total aircraft weight on the panel at one time or provided the most eccentric loading. Configurations with multiple wheel loads on the panel at one time were converted to a composite eccentric load at some point on the panel by summing the moments about both directions (x-and y-axis) from the center of the panel. The effect of the panel weight was also considered and added to the calculations for a more accurate estimate. The panel added significant load to the underlying soil even though it was centered and made the situation less eccentric. Calculations of the panel weight alone could also be used to determine the flowable fill strength required to remove installed bridge plates so that the panel can be self supported.

The results of the bearing capacity calculations completed for seven loading configurations are shown in Table 9. Cases 4 and 6 are the most damaging for C-17 and F-15E traffic, respectively. The calculations indicated that the minimum compressive strengths required for F-15E traffic and C-17 traffic were 26 and 41 lb/in.^{2}, respectively. However, a safety factor of 2 was recommended for implementation; therefore, the flowable fill should achieve at least 55 lb/in.^{2} prior to supporting F-15E traffic and 85 lb/in.^{2} for C-17 traffic (when rounded to the nearest 5 lb/in.^{2}).

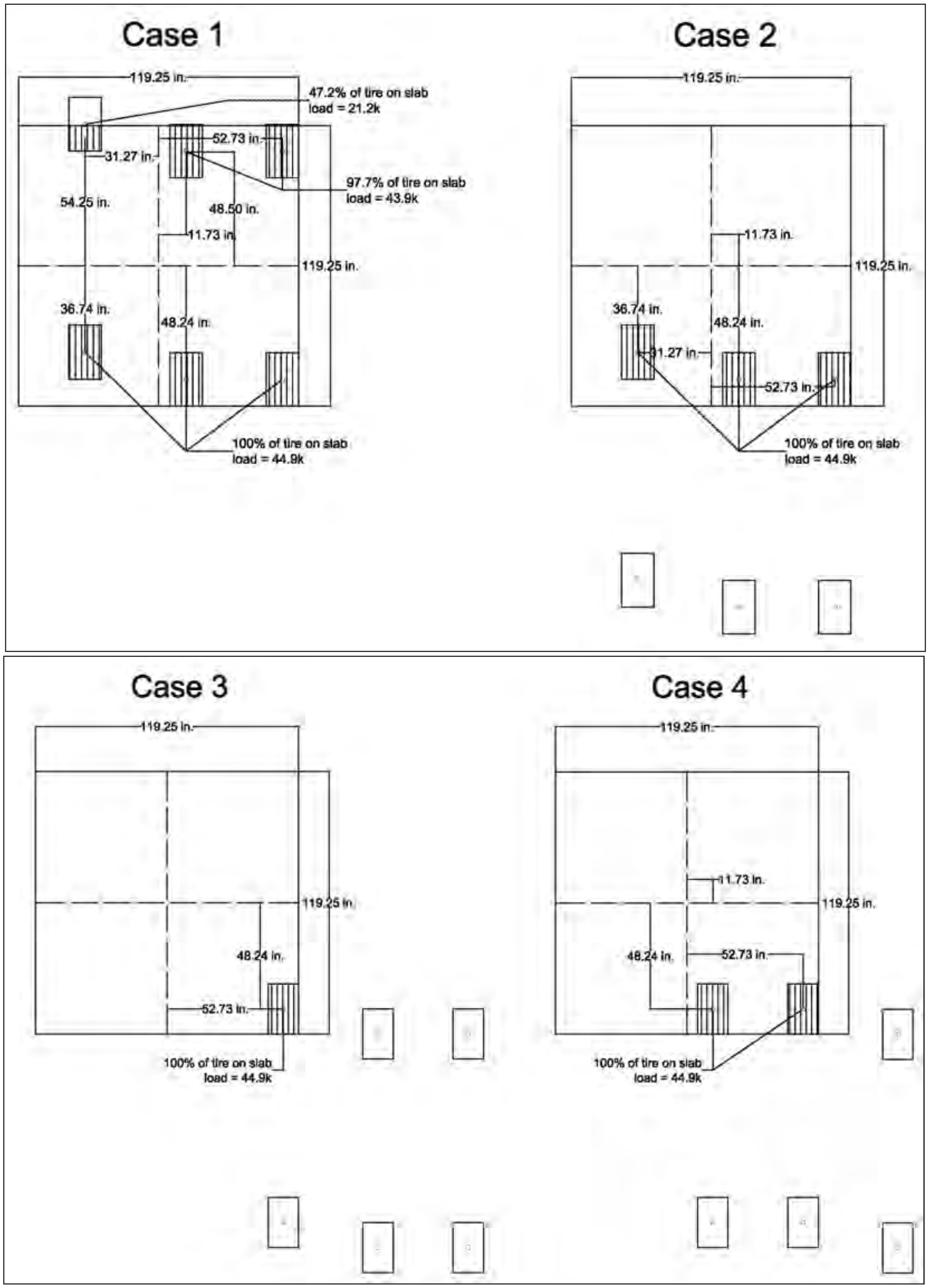

Figure 46. Critical C-17 aircraft loading scenarios for installed precast panel.

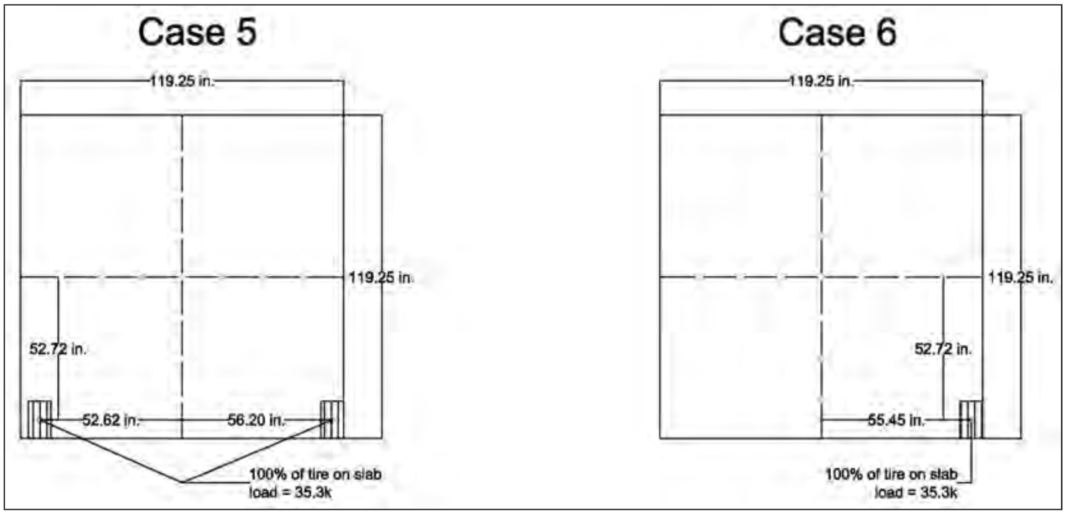

Figure 47.Critical F-15E aircraft loading scenarios for installed precast panel.

Table 9. Calculated bearing capacity needs for panel trafficking.

Case	Aircraft		Composite Loading			Effective area, in.^{2}	Bearing capacity (safety factor), lb/in.^{2}	
	Type Considered	Main gear wheels considered	load, kip	Eccentricity from center			Minimum	Recommended
				x, in.	y, in.			
Panel Only	-	-	13.6	0	0	14,221	1	5 (5)
1	C-17	6	257.3	14.2	-2.2	12,443	21	41 (2)
2		3	148.3	10.1	-40.3	4,764	31	61 (2)
3		1	58.5	40.5	-37	1,945	30	60 (2)
4		2	103.4	28.0	-41.9	2,522	41	82 (2)
5	F-15E	2	84.2	1.2	-44.2	3,911	22	43 (2)
6		1	48.9	40.1	-38.1	1,899	26	52 (2)

One loading case not discussed in detail was the bearing capacity required to support a precast panel alone before applying aircraft traffic. This situation was expected to be very plausible for field applications where multiple panels are placed within a repair window under limited bridge plate supplies. Removing bridge plates after the flowable fill gains sufficient strength to support a panel alone, frees bridge plates for additional installations elsewhere and increases the total number of repairs capable for completion in the repair window. Each panel's large plan area places minimal load on the flowable fill, and requires little bearing capacity for support in this case. A safety factor of 5 was recommended for the bearing capacity in this scenario to account for delivered material differences at required bearing capacities this low.

Additional flowable fill material requirements

Material strength was not the only factor considered when selecting the flowable fill requirements. For efficient placement and quick reopening times, considerations for the workability and rate of strength gain were also required for the system to perform as needed. Time requirements for the minimum compressive strength at reopening were variable and depended on the speed of the operations required. For speedy reopening times, approximately between 4 and 8 hr, typically flowable fill mixture designs utilizing conventional materials would not provide the strength gain needed. Removing water from the mixture could aid strength gain, but typically reduces material workability required for efficient placement.

Additives or alternative materials would be required for shorter reopening times. Calcium chloride salt is an effective and common additive used to accelerate strength gain of conventional or field prepared flowable fill batched with portland cement. Dosages of less than 3 percent by weight of cementitious material were recommended by the ERDC to provide the projected wet working times required to repair damaged airfield pavement. Proprietary rapid-setting flowable fill materials are also commercially available and can provide strengths greater than 200 lb/in.^{2} within 4 hr. Additional admixture chemicals may be needed for hot or cold weather operations in addition to any air entrainment or water reducing additives, if not provided.

Efficiently placing flowable fill in the repair area such that the contours of the repair surface are completely filled, there are no voids within the placed material, and the panel is adequately seated are all essential to the performance under traffic. Previous work conducted by the ERDC using flowable fill to backfill airfield repairs under rapid-setting concrete caps recommended a flowability of 10 ± 1 in. by ASTM D6103 for successful repairs. Since the installed precast repair is very similar to the cast in place rapid-setting repair, using similar guidance is expected to yield similar results.

5 Fabrication of Precast Panels

During August through October 2011, ERDC and AFRL personnel cast 3 terminal and 7 standard panels. This section describes the general steps in the process for preparing the panels. Detailed instructions and photographs of the fabrication process are provided in Appendix A.

Step 1: Secure construction and storage areas

For this project, panels were prepared in an open-ended aircraft hanger, Hangar 2, located at the ERDC in Vicksburg, MS. This location provided a construction area with a smooth, level, hardened pavement surface with water and electrical hookups, and had a sound pavement upon which to secure the panel formwork during placement of the PCC. Securing the forms to a hard surface such as a pavement is required to prevent movement of the forms to ensure panel dimensions meet required tolerances. The area was also large enough to construct all panels without removing the panels right away and had sufficient overhead clearance to operate the crane while still providing a shelter from the environment.

Step 2: Obtain equipment and materials for fabrication

All materials required for constructing the panels were gathered and moved to the preparation area. All debris was removed from a previously constructed 14-in.-thick PCC test section selected to be used for construction/ fabrication of the panels. Concrete was ordered from a local vendor, meeting the requirements presented in Chapter 4, and the mixture design was provided in Figure 48. Once concrete was placed in the panel forms and finished, they were sprayed with curing compound and were allowed to cure for at least 28 days.

Each panel required approximately 4 yd^3 of concrete assuming 15 percent waste. 8 yd^3 of concrete was ordered to prepare two panels at a time. The average 28-day compressive results were 5,710 $lb/in.^2$, which is above the minimum 5,000 $lb/in.^2$ compressive strength at 28 days specified. Table 10 summarizes the strength test data for the PCC panels.

Material Properties and Source

Cementitious Material	Type	Source	Specific Gravity
Portland Cement	II	Holcim	3.15
Fly Ash	C	Headwaters	2.59
GGBFS (Slag)			

Admixtures	Name	Supplier	Dosage, Fl. Oz.
Type A	322 N	BASF	1-3 per cwt.
Type F	7500	BASF	4-8 per cwt
AE	MB90	BASF	3% - 6%

Note: Dosage rate will require adjustments for field and environmental conditions.

Aggregate Size	Type	Supplier	Sp. Gr. SSD	Sp. Gr. OD	Absorption, %	F.M.
# 57	Stone	Vulcan	2.68	2.67	0.80	
Sand	Natural	Green Bro	2.60	2.58	0.66	2.65

Batch Quantities

Material	Quantities lb/yd^3 SSD	Absolute Volume ft^3
Cement, lb.	489	2.49
Fly Ash, lb	122	0.75
Mix Water, lb.	245	3.93
Slag, lb.		
Coarse Aggr., lb.	1850	11.06
Fine Aggr., lb.	1225	7.55
Air Content, %	4.5	1.22
Total Mass, lb.	3931	27.00

Water / cementitious material ratio: 0.40

Mix Design Information:

Mix Class: 5000 psi. with Air

Comments: 650 Flex

Designed by: Andrew Lester

Title: Regional QA Manager

Organization: MMC Materials

Figure 48. PCC mixture used for precast panels.

Table 10. PCC test data.

Panel #	Specimen Number	Type of Specimen	Age in Days	Compressive Strength, lb/in.^{2}
1	1	4x8 cylinder	28	5,750
1	2	4x8 cylinder	28	5,700
Average				5,730
2	1	4x8 cylinder	28	6,040
2	2	4x8 cylinder	28	6,000
Average				6,020
3	1	4x8 cylinder	28	6,800
3	2	4x8 cylinder	28	6,630
Average				6,715
4	1	4x8 cylinder	28	6,200
4	2	4x8 cylinder	28	6,670

Panel #	Specimen Number	Type of Specimen	Age in Days	Compressive Strength, lb/in.^{2}
Average				6,440
5	1	4x8 cylinder	28	6,010
5	2	4x8 cylinder	28	5,940
Average				5,980
6	1	4x8 cylinder	28	6,000
6	2	4x8 cylinder	28	6,000
Average				6,000
7	1	4x8 cylinder	28	5,530
7	2	4x8 cylinder	28	5,880
Average				5,710

Step 3: Assemble formwork

The forms were organized according to the etched labels on each of the form pieces allowing 20 ft between form setups to provide working space around the forms during fabrication and to allow room to remove the forms after the panels had cured. To assemble a form, the corners were aligned and connected using the key slots in the forms. Then, 0.75-in. diameter bolts were inserted into the holes on the form ends and were tightened using a hand ratchet. The process resulted in a square form. Once assembled, the squareness of the forms was checked by measuring each diagonal, and the tops of the forms were checked for flushness.

The next step was to secure the forms to the pavement. Securing the forms to a sound pavement is necessary to prevent the forms from floating and to maintain form position during concrete placement. Concrete anchors (0.5-in. diameter and 7-in. long) were used to secure the steel form to the pavement using concrete anchors through tabs centered on the bottom edge of each segment of formwork. Anchor holes were drilled using a 0.5-in. masonry drill bit and hammer drill to a depth of 6 in. Concrete anchors were then installed and driven into the pavement using a steel mallet. After the installation of a single anchor, the forms were checked for squareness. Once all anchors were in place, the concrete anchors were tightened using an electric impact wrench. Debris removed during the drilling process was removed using a shovel or broom.

Following the assembly of the forms, the forms were sprayed with form release oil to prevent bonding of the precast panel. During the spraying of release oil, all interior and exterior surfaces including connection bolts were also sprayed to facilitate clean up and prevent concrete build up. A thin sheet of plastic was also inserted in the bottom of the form as a barrier between the fresh concrete and the underlying pavement.

Step 4: Install reinforcement

Once the form was prepared, a steel reinforcement grid was installed. The reinforcement grid consisted of grade 60, 0.375-in.-diameter rebar (#3), arranged in a 1-ft-square pattern. Grade 60, 0.625-in. rebar (#5) was also utilized. The reinforcement layouts for both panel types were shown previously in Figures 31 and 32. A 1.5-in. concrete cover was maintained between the ends of the reinforcement bar and the side of the frame. The entire grid was placed on 1.5-in. rebar chairs to provide the cover between the reinforcement and a cover of 1.5 in. between the reinforcement and the bottom of the precast panels. The following paragraphs describe this process in more detail.

After positioning and securing the frame to a PCC surface, 18 #3 bars were used to prepare a reinforcement grid. For this project, the rebar was specially ordered precut to a length of 9 ft 7.5 in. ± 0.5 in. The rebar was measured to ensure that the materials were the proper length. Any pieces of rebar that were too long were trimmed, and any rebar that were too short were not used.

The rebar was marked using spray paint placing the first mark 9.875 in. from the bars' ends and then marked in 12-in. intervals. Nine of the bars were then spaced approximately 12 in. apart (on center) on a smooth flat surface and overlapped with the remaining nine bars in the opposite direction to layout the grid. Each piece of rebar was aligned so that the paint marks prepared earlier overlapped. Then 6-in. precut rebar ties were used to secure the rebar tightly together using ratchet tying tools. All exterior connections were secured first, then interior connections were performed in a checkerboard pattern (i.e., not all connections in the interior portion of the grids were tied).

Once the main reinforcement grid was complete, a layer of #5 reinforcement bars was attached to the existing #3 rebar grid. Four pieces of #5 rebar, each measuring approximately 9 ft 7.5 in. in length, were tied to the top of the existing #3 rebar grid (2 #5 bars placed at each transverse end perpendicular to the load transfer dowels).

The entire reinforcement grid was then lifted and placed within the constructed formwork such that the #5 bars were oriented perpendicular to the load transfer dowels. The grid was centered within the formwork to allow a 1.5-in. gap between the ends of the reinforcement and the formwork. Then, approximately 25 1.5-in. plastic bar chairs were placed under the lower #3 bar layer of the grid to hold the reinforcing grid at the correct elevation to allow the proper cover of the bottom of the panel to the reinforcement (1.5 in.). The chairs were spaced evenly to minimize any sagging and provide sufficient stability.

Step 5: Install load transfer dowels

The next step was to insert load transfer dowels into the forms. The standard form was designed with dowel openings on two opposing sides of the panel. The terminal form was designed with dowel openings on only one side of the panel. The dowels were centered 5.5 in. from the top surface of the panel. Each 1-in.-diameter dowel was 22 in. long. This length allowed 11 in. for the precast panel, with an additional 11 in. remaining to tie into the parent slab. Dowel rods were spaced 1 ft apart with the first dowel situated 6 in. from the edge of the precast panel. Exterior dowel alignment forms were used to maintain proper alignment during concrete placement.

Ensuring and maintaining proper alignment of each dowel was imperative. Misaligned dowels have the potential to prevent form removal, may damage the precast panel and/or the rigid form, and could negatively impact load transfer and/or cause premature pavement failure. The following paragraph describes the dowel installation process in greater detail.

Each dowel center was marked and taped with duct tape. The tape was necessary to help seal the void around the dowel openings in the frame. Each dowel was then lightly greased on one end and installed into the formwork by sliding the non-greased end from the interior of the formwork through the dowel receptacle. Inserting the dowels in this direction eliminated the loss of grease. Once all dowels were installed, the dowel receptacles were packed with grease to prevent concrete from flowing through the gap between the dowels and the edges of the dowel receptacles.

Step 6: Install blockouts (terminal panel only)

To cast dowel receptacles in the terminal panels, blockouts were installed on the side of the panel opposite the dowels. Prior to installation, all threaded holes and bolts were greased to ensure concrete did not flow around the threads. Additionally, form release oil was applied to all surfaces of the blockouts. The blockouts were then bolted to the form using individual bolts. For ease of installation, the top surface was stamped “TOP”. After ensuring that the top of each blockout was parallel and level with the top of the form, all bolts were tightened with an impact wrench.

Step 7: Install the upper #5 reinforcement bars

Following the installation of dowels on both forms and the installation of the blockouts on the terminal form, the next step was to install an upper layer of #5 reinforcement bars. Two groups of three bars were installed, and their installation location depended on the dowel bar arrangements on the side of the form.

For the dowel sides of forms:

Two of the #5 bars were positioned on top of the installed dowels, located at 5 and 10 in. on center from the edge of the formwork (vertically above and parallel to the #5 bar installed on reinforcement grid). A third bar was then placed with its center 15 in. from the edge of the formwork. This bar was held in place using three 6-in.-high reinforcing chairs to support the bar. The #5 bars were then tied to the dowels or chair. When tying to the dowels, 8-in.-long ties were used; otherwise 6-in.-long ties were used. Once tied, the ends of the #5 bars were checked to be flush with one another and the underlying #3 bars. (If correctly placed, the #5 reinforcement were perpendicular to the load transfer dowels.) Then 1-ft-long sections of #3 rebar were tied underneath the upper #5 bars. A total of 3 of these sections were equally spaced and centered between the original layers of #3 bars. Each was tied at the intersections.

For the terminal form:

Since there are no preinstalled dowels at this location, all bars were supported using 6 in.-high rebar chairs. Bars were installed 14, 19, and 24 in. on center from the edge of the formwork. These distances were measured perpendicular to the exterior most blockout. Each #5 bar was placed on 6-in.-high chairs to which they were tied.

Step 8: Place PCC

Following preparation of the forms, each was filled with PCC as specified in the previous chapter. The concrete was placed, consolidated, screeded, and finished following typical concrete placement techniques. A 1.5-in. diameter spud vibrator was used to consolidate the concrete. Particular care was taken to ensure good consolidation around the edges, in corners, around blockouts and dowels. Vibration was essential to prevent damage and yield the proper panel shapes. Each panel was slightly overfilled by 0.5 in. to provide enough material to fill in the surface while screeding. The panels were screeded using a vibratory concrete screed, and excess material was removed during screeding as needed to provide a PCC surface that was flush with the top of the forms. A 4-ft bull float and magnesium hand floats were then used to level and fill the slab surface. Care was taken not to over finish the surface. No water was used in finishing the panels, and finishing edgers were not used to round edges.

Following concrete placement, all materials used to place, consolidate, and finish the slabs were cleaned and the dowel bar embedment depth was checked for each panel. If necessary, the dowels were adjusted.

Step 9: Install lifting anchors

Just prior to placing the PCC, the lifting anchors were prepared by assembling the anchor and attaching a rubber recess form (provided with the lifting anchor by the manufacturer) around the top (thicker) end of the anchor. Then duct tape was used to completely seal the seams and surface of the rubber recess form to prevent mortar from entering during concrete placement.

After the PCC had been placed, lifting anchors were installed into the PCC approximately 30 to 60 min after finishing to allow the concrete to stiffen and prevent the anchor from settling beneath the surface. A plywood template was used to aid in placing the anchors 2.5 ft from the corner of each panel. The anchor was inserted until approximately 0.25 in. of the rubber recess form was above the surface of the PCC. Then a hand float was used to fill any holes around each anchor and to push the anchor into position where it was flush with the surface of the surrounding concrete.

Step 10: Final finish

After the lifting anchors had been installed and the concrete had stiffened sufficiently, final finishing was completed with the bull float to remove any excess water and surface deformations caused from the smaller hand finishing tools. A broom finish was then applied to each panel. Panels were then coated with a double coat of curing compound using a hand pump sprayer.

Step 11: Form removal

After approximately 7 days, the concrete forms were removed from the panels. The anchor bolts holding the forms to the underlying concrete pavement were first removed. Then, the connection bolts holding the forms together were removed. A mallet was used to remove the longitudinal form sections to break the bond between the PCC and the form by striking the ends of each corner of the form. Care was taken to remove the transverse form sections to not damage the dowels or the dowel blockouts. Lumber was used to pry the form segments away from the PCC. Using lumber mitigates potential damage to the precast slab (i.e., chipping, spalling).

For forms using dowel block outs, the bolts connecting the blockouts to the form were removed, and the block outs remained in the PCC. The bolts were then reinserted into the block outs to provide leverage to remove each blockout by lifting upwards. A hammer was used if needed to help in removal.

Once the forms were removed, the tape surrounding the tops of the lifting anchors was removed. All surfaces that had been covered with the forms were then sprayed with curing compound and allowed to continue to cure in place for a total of 28 days. Forms were then reassembled in a different location, and all steps were repeated until adequate numbers of panels had been prepared.

6 Test section construction and field testing

Test section construction

General design

A 60-ft-wide x 100-ft-long PCC test section was constructed at the ERDC in Vicksburg, MS during the period April through July 2011. The test section was designed using the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) software. Assumptions used as input to the software were: a PCC airfield surface with a flexural strength of 650 lb/in.^{2}, no drainage layer, an aggregate base thickness of 6 in., a modulus of subgrade reaction of 150 lb/in^3, and a design life of 50,000 C-17 passes. Using the PCASE software and the above assumptions, a 14-in. thick PCC layer was required. The software recommended a pavement joint spacing of 15 to 20 ft with dowel spacing of 15 in., dowel length of 20 in., and a dowel diameter of 1 to 1.25 in. Based on these results, the PCC test section was designed to consist of 15 20-ft-wide x 20-ft-long slabs, each 14-in. thick as shown in Figure 49. Dowel diameter was selected to be 1 in. with the required length of 20 in. and spacing of 15 in.

Seven of the ten precast repair panels fabricated were installed in the test section as shown in Figure 50. The precast panels were placed over a flowable fill backfill material. The remainder of this chapter details the construction of the full-scale test section.

Sublayer preparation

The subgrade of the test section was prepared by excavating all surface material to a depth ranging from 20 to 22 in. The natural soil consisted of low-plasticity clay with sand with a soil classification of CL. The material had a liquid limit of 36 and a plasticity index of 14. The material consisted of 8.7 percent gravel, 11.5 percent sand and 79.8 percent fines. Modified proctor testing resulted in a maximum dry density of 120.7 lb/ft^3 and an optimum moisture content of 12.1 percent. The CL was compacted with 3 passes of a 23,150-lb vibratory smooth drum compactor to create a firm working platform for placement of the upper foundation materials.

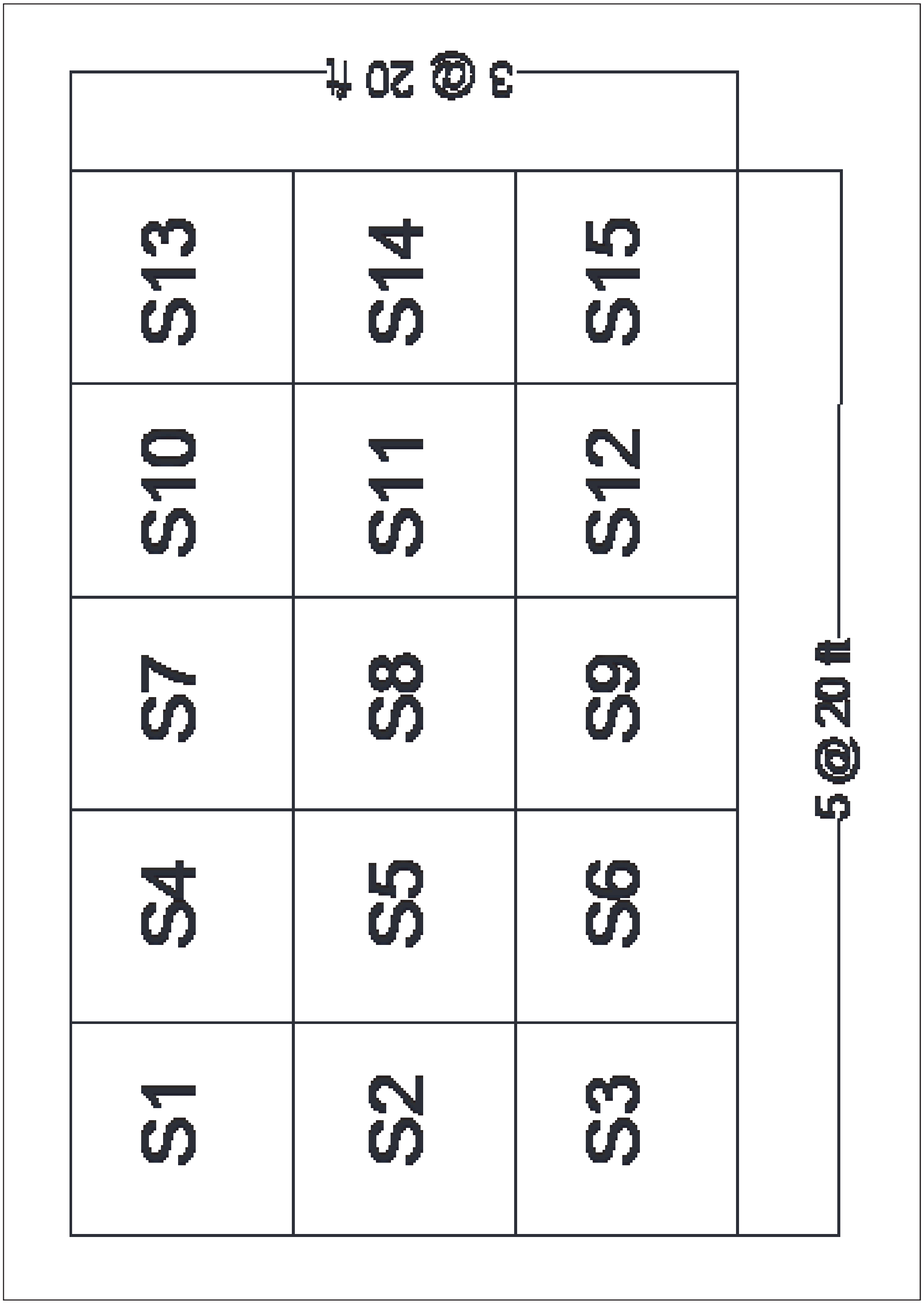

Figure 49. PCC test section original construction layout.

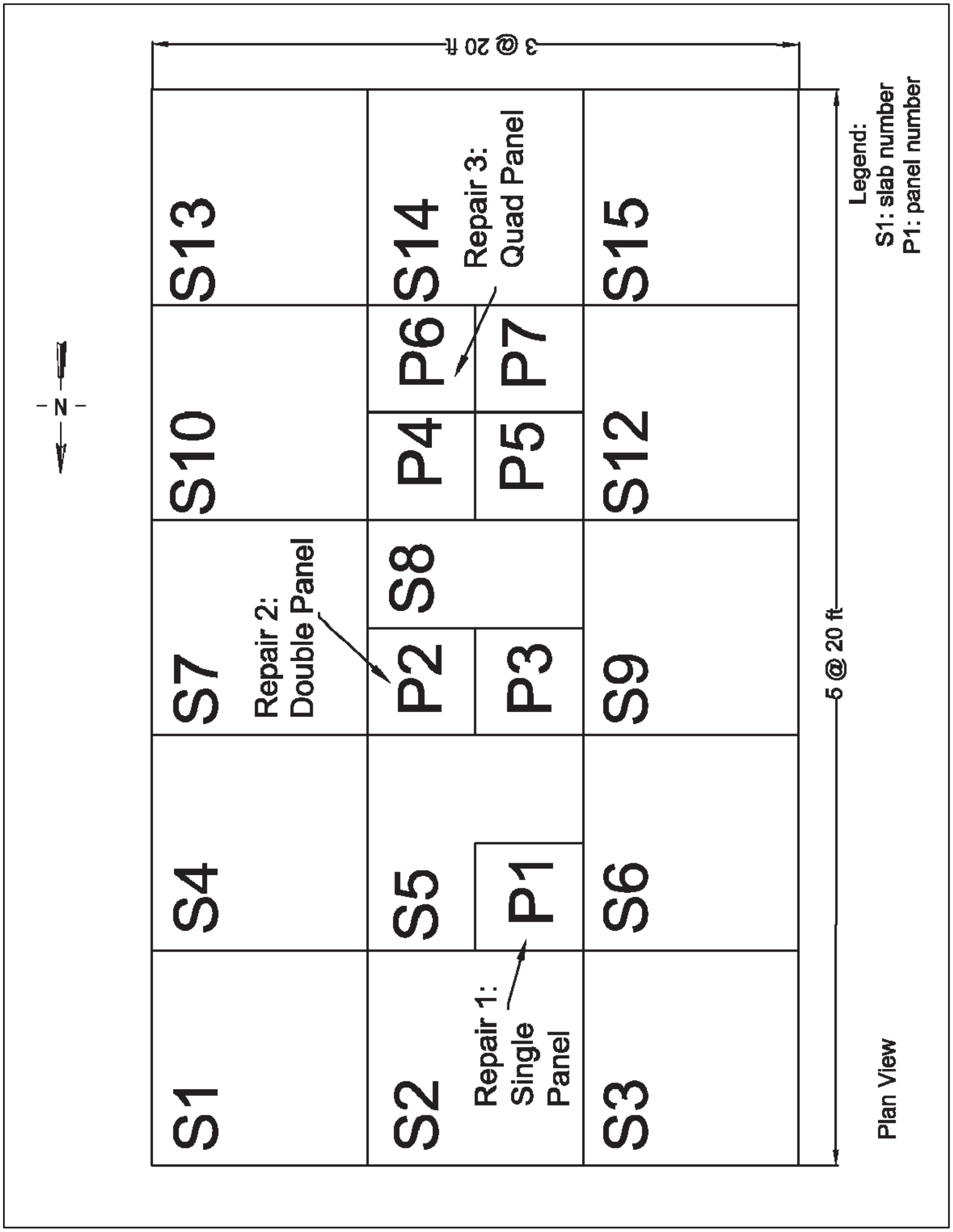

Figure 50. PCC repair panel layout-plan view.

A 6-in. thick base course was then constructed using crushed limestone with a classification of GW. The base was compacted with 3 passes of a 23,150-lb vibratory smooth drum compactor. Both subgrade and base materials were placed on a 1 percent (1 ft) longitudinal slope and a 0.5 percent (0.3 ft) cross slope for drainage.

The foundation materials were further characterized by performing dynamic cone penetrometer (DCP), nuclear moisture-density, and modulus of subgrade reaction tests after compaction was completed. Table 11 summarizes pertinent soils test data for the sublayers. As seen in the table, the DCP estimated CBR under slab 1 was much stronger than the other test areas. The DCP test results indicated that the base layer under this slab was stronger than the other test points in the test section. This is either because of better compaction in this area, or the DCP encountered a rock. Plate load testing conducted on the surface of the base course resulted in an effective modulus of subgrade reaction *(k)* of 276.

Table 11. Sublayer soils test data.

Corresponding Slab	Location	Moisture, percent	Dry Density, lb/ft^3	CBR, percent	Effective Modulus of Subgrade Reaction, lb/in.^{2}/in.
S1	Subgrade	10.3	116.4	31	276
S1	Base	2.1	127.5	95	
S3	Subgrade	11.6	108.5	25	
S3	Base	1.3	123.7	29	
S7	Subgrade	12.7	112.2	9	
S7	Base	2.7	126.3	15	
S9	Subgrade	9.0	115.7	6	
S9	Base	1.6	121.8	33	
S13	Subgrade	3.9	128.5	9	
S13	Base	4.9	124.2	27	
S15	Subgrade	12.6	113.8	6	
S15	Base	2.0	125.4	21	

PCC placement

Concrete construction work was completed in July 2011 by ERDC's Directorate of Public Works and APB personnel. The test section dimensions were 100 ft x 60 ft x 14 in. A fixed-form placement was completed using wooden formwork constructed at ERDC. The joints were doweled using 1-in.-diameter, 20-in.-long epoxy coated steel dowels as shown in Figure 51. Dowels were spaced 15 in. on center and placed mid-depth of the slab (7 in.). Dowel were installed into the concrete by pre-placing the dowels through holes in the formwork before placement. Concrete was placed in the two outer lanes of the test section followed by filling in the interior portion. A locally available 650- lb/in.^{2} flexural strength fixed form paving concrete was used. Following placement, a light broom finish was applied to the section followed by a double coating of white pigmented membrane forming curing compound. The average 28-day compressive and flexural strength from field cured samples was 940 lb/in.^{2}, well above the minimum design strength. Table 12 summarizes the strength test data for the PCC.

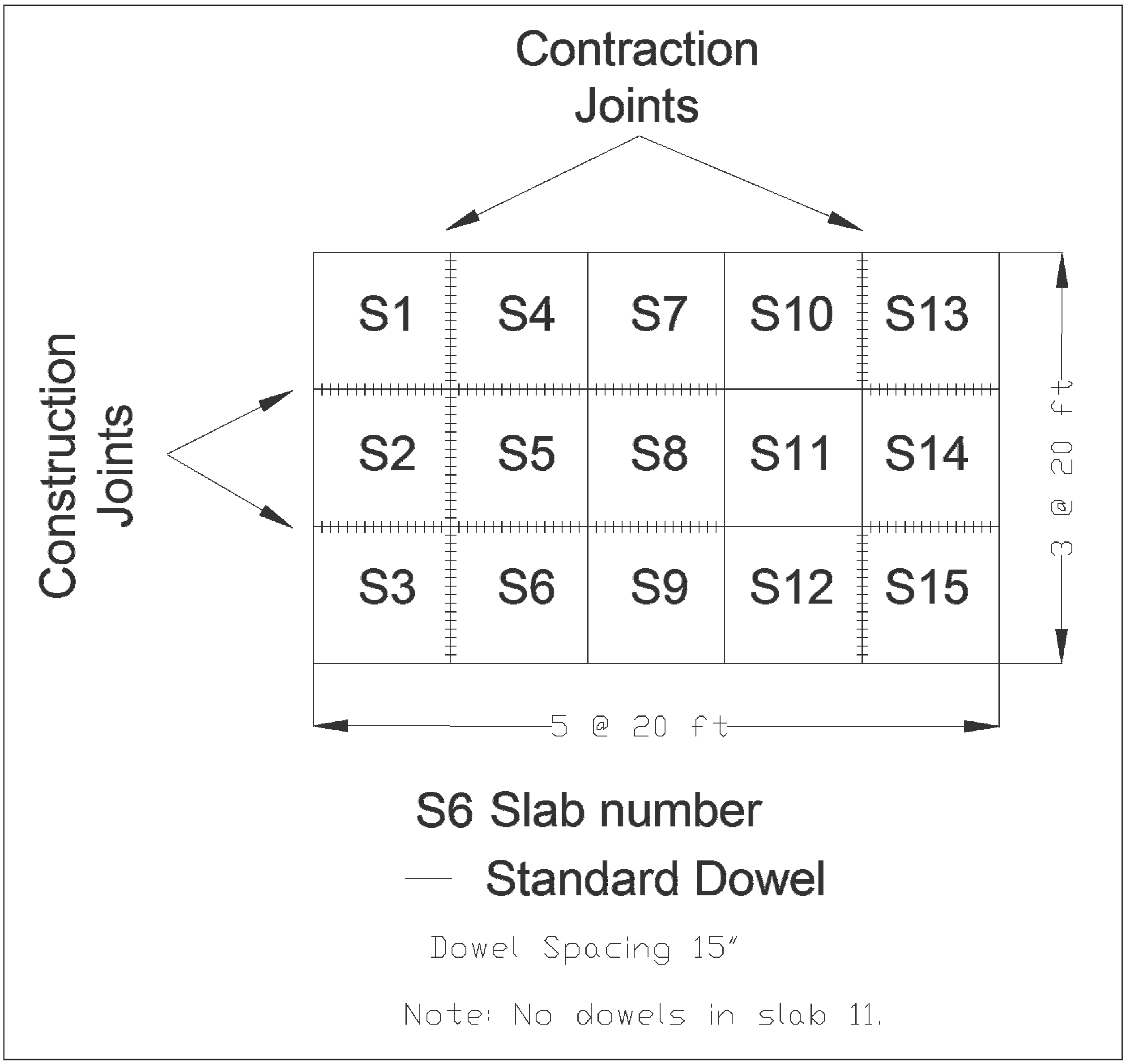

Figure 51. Dowel locations.

Table 12. PCC test data.

Truck Number	Specimen Number	Type of Specimen	Age in Days	Compressive Strength, lb/in.^{2}	Flexural Strength, lb/in.^{2}
Lane 1 Truck 3	1	6x12 cylinder	28	6,950	--
	2	6x12 cylinder	28	7,200	--
	3	6x12 cylinder	28	6,870	--
	4	Flex Beam	28	--	850
	5	Flex Beam	28	--	840
	6	Flex Beam	28	--	840
	Average			7,010	840
Lane 1 Truck 5	7	6x12 cylinder	28	5,840	--
	8	6x12 cylinder	28	6,340	--
	9	6x12 cylinder	28	6,280	--
	10	Flex Beam	28	--	720
	11	Flex Beam	28	--	770
	12	Flex Beam	28	--	740
	Average			6,150	740
Lane 1 Truck 8	13	6x12 cylinder	28	6,250	--
	14	6x12 cylinder	28	6,590	--
	15	6x12 cylinder	28	6,190	--
	16	Flex Beam	28	--	750
	17	Flex Beam	28	--	760
	18	Flex Beam	28	--	780
	Average			6,340	760
Lane 2 Truck 3	1	6x12 cylinder	28	7,930	--
	2	6x12 cylinder	28	6,670	--
	3	6x12 cylinder	28	6,630	--
	4	Flex Beam	28	--	1,060
	5	Flex Beam	28	--	980
	6	Flex Beam	28	--	1,000
	Average			7,080	1,010
Lane 2 Truck 5	7	6x12 cylinder	28	8,390	--
	8	6x12 cylinder	28	7,930	--
	9	6x12 cylinder	28	8,890	--
	10	Flex Beam	28	--	960

Truck Number	Specimen Number	Type of Specimen	Age in Days	Compressive Strength, lb/in.^{2}	Flexural Strength, lb/in.^{2}
	11	Flex Beam	28	--	930
	12	Flex Beam	28	--	1,060
	Average			8,400	980
Lane 2 Truck 5	13	6x12 cylinder	28	8,200	--
	14	6x12 cylinder	28	7,000	--
	15	6x12 cylinder	28	8,160	--
	16	Flex Beam	28	--	1,110
	17	Flex Beam	28	--	810
	18	Flex Beam	28	--	1,030
	Average			7,790	980
Lane 3 Truck 8	1	6x12 cylinder	28	7,240	--
	2	6x12 cylinder	28	7,620	--
	3	6x12 cylinder	28	7,650	--
	4	Flex Beam	28	--	1,040
	5	Flex Beam	28	--	1,160
	6	Flex Beam	28	--	1,310
	Average			7,500	1,170
Lane 3 Truck 6	7	6x12 cylinder	28	7,500	--
	8	6x12 cylinder	28	7,550	--
	9	6x12 cylinder	28	7,630	--
	10	Flex Beam	28	--	1,040
	11	Flex Beam	28	--	960
	12	Flex Beam	28	--	980
	Average			7,560	990
Lane 3 Truck 12	13	6x12 cylinder	28	5930	--
	14	6x12 cylinder	28	8020	--
	15	6x12 cylinder	28	7960	--
	16	Flex Beam	28	--	990
	17	Flex Beam	28	--	990
	18	Flex Beam	28	--	1,020
	Average			7,300	1,000
Average				7,240	940

After curing for 6 hr, joints were sawed to a depth of 4.5 in. A 0.5-in. wide x 3/8-in. deep back cut was made to create a joint sealant reservoir. The joints were sealed by first installing a layer of bond breaking tape applied to the depth of the cut and a ¼-in. thick layer of field molded joint sealant a few days following cutting the back cuts to allow the concrete to dry.

Field testing- precast panel repairs

Following curing for 28 days, three areas were marked in the test section to represent damaged pavement requiring repair as shown in Figure 50. Repair 1 simulated a partial slab replacement along two joints with a single-panel repair. This repair surface area was 10 ft x 10 ft. Repair 2 simulated a half slab replacement along a joint requiring two panels to repair a surface area of 10 ft x 20 ft. Repair 3 simulated a full slab replacement requiring 4 panels to repair a surface area 20 ft x 20 ft. The following sections describe the repair process for the three repair areas. Equipment and materials required to complete repairs is listed in Table 13.

Table 13. Equipment and materials required for placing precast panels.

Equipment/Supplies	Activity							
	Delivery	Distressed Area Prep	Distressed Area Removal	Bridge Plate Installation	Base Preparation	Panel Placement	Dowel Sleeve Sealing	Joint Sealing
4-ton Swift Lifting Eyes	x		x			x		
5-gal Plastic Buckets							x	
8 ft Synthetic Endless Round Slings	x		x			x		
Air Compressor			x	x			x	x
Backer Rod								x
Dowel Receptacle Template		x						
Bolts, Washers, etc.			x	x				
Bridge Plates (custom)				x		x		
Bridge Plate Template (custom)								
Bucket Opener (for dowel grout)							x	
Chalk Line Tools		x						
Concrete Anchor Drilling Jigs		x		x				
Concrete Expansion Anchors		x						

Equipment/Supplies	Activity							
	Delivery	Distressed Area Prep	Distressed Area Removal	Bridge Plate Installation	Base Preparation	Panel Placement	Dowel Sleeve Sealing	Joint Sealing
Concrete Floor Saw and Blades		x						
Concrete Sand						x		
Drilling Hammers		x		x				
Dunnage	x							
Expansion Joint Board							x	x
Extension Cords		x		x			x	x
Flatbed Truck	x							
Floor Brooms		x		X	X	x	x	x
Flowable Fill in Concrete Truck					x			
Flowable Fill Screed (custom)					x			
Generators		x		x			x	x
Grout Hand Mixers with Paddles							x	
Hand Elements- Shovels, Concrete Rakes, Pry Bars, Hammers, etc.			x	x	x	x		
Hand Floats							x	
Hand Grinder				x				
Heavy Duty Garden Hoses					x		x	
Impact Hammer		x						
Infrared Thermometer							x	x
Joint Sealant Gun								x
Knee Pads				x	x	x	x	
Lifting Eyes	x	x	x			x		
Marking Crayons/Paints		x		x	x			
Measuring Tapes		x		x	x	x		
Crane	x		x			x		
Pavement Breaker Chisels			x					
Pavement Breakers			x					
Pedestal (for 4-panel repair)						x		
Various Wrenches	x	x	x	x				
Plywood						x	x	
Portable Band Saw						x		

Equipment/Supplies	Activity							
	Delivery	Distressed Area Prep	Distressed Area Removal	Bridge Plate Installation	Base Preparation	Panel Placement	Dowel Sleeve Sealing	Joint Sealing
Personal Protective Equipment	x	x	x	x	x	x	x	x
Precast Panels	x					x		
Rapid-Setting Repair Mortar							x	
Silicon Joint Sealant								x
Shop Vacuum			x					
Swivel Hoist Rig	x		x			x		
Torque Wrenches		x						
Various Thickness Shims						x		
Vibratory Roller						x		

A team of 12 people was used for the installation although not all of them were used for every repair step. Numerous activities were conducted simultaneously with subteams consisting of 2 to 3 people. One of the team members was a licensed crane operator needed to remove the damaged pavement and place the precast panels in place. Additional details and photographs of placement are provided in Appendix B.

Repair 1- single-panel repair

Repair 1 was conducted during early November 2011. The following sections detail the process and steps required for conducting this single-panel repair.

Distressed area removal

The process for removing the distressed area is presented in the following steps and in photographs in Figure 52.

1. Mark area for removal

   For Repair 1, a 10-ft x 10-ft section of Slab 5 as shown in Figure 50 was selected for removal. A chalk line tool was used to delineate the area to be removed and waterproof paint was applied to mark the saw cut perimeters. After the repair boundary area was prepared, locations of

Figure 52. Preparation and removal of parent PCC: a) saw cutting dowel receptacles and repair area; b) drilling to install concrete lifting anchors; c) installing lifting eyes; d) connecting lifting eyes to crane rigging; e) lifting “damaged” pavement; f) preparation of dowel receptacles using jackhammers; g) close up of dowel receptacles; and h) prepared repair area.

dowel receptacles were then marked using a plywood template and spray painted. For a single-panel repair, dowel receptacles were marked on both sides of the repair area in the direction of traffic.

2. Perform saw cutting

   Once all locations for pavement removal and dowel receptacles were marked saw cutting operations were conducted using a concrete saw capable of cutting a depth of at least 18 in. The square repair area was saw cut first. The saw cutting operations were completed using a series of 4 passes, with the depth of cut being progressively increased approximately one-quarter of the total slab depth during each pass. Progressively larger saw blades were utilized to ensure precise, accurate cuts. The first and second passes were each made with a 24-in. diameter blade. A 36-in.-diameter blade was used for the third and fourth (final) passes.

   A single 36-in.-diameter saw blade could have been used to make each progressive pass, or the saw cutting could have been completed with a single pass. However, this is not recommended due to excessive saw kerf potential; which could be problematic during panel installation. Intermediate blade diameters should be used when possible. However, it is time consuming to change the blades, which can impact the expediency of the repairs.

   Once the 10-ft x 10-ft area was saw cut, the dowel receptacles in the parent concrete pavement were saw cut. Dowel receptacles should be cut prior to extracting the damaged slab to prevent saw operations near a pavement void. Each marked dowel receptacle was 12-in. long, 3-in. wide, and was designed to be 6 ½ -to 7-in. deep after excavation. Two 6.75-in. deep vertical cuts were made for each dowel location. The saw operator used a 24-in.-diameter blade to cut the dowel receptacles. After setting the proper blade depth, the saw was inserted into the PCC such that the center of the blade was plunged into the existing pavement (section not designated for removal) at the back of the marked dowel receptacle (12 in. from the perimeter of the saw cut slab). Individual dowel receptacle saw cutting operations were terminated when the center of the saw blade was centered at the joint marking the perimeter of the damaged slab. This step is shown in Figure 52a.

3. Install expansion anchors

   After saw cutting, expansion anchors were installed to remove the parent PCC as a monolithic section. To install the anchors, each anchor location was marked using paint 30 in. from each corner of the slab to be removed. Four anchors were required for balance and slab stability when lifting.

   A rotary hammer and a 1.25-in. concrete drill bit were then used to drill 12-in. deep holes in which the expansion anchors would be installed. This step is shown in Figure 52b. Care was taken to ensure that the drill bit remained fairly vertical to ensure swift installation of the anchors. Once the holes were drilled, compressed air was used to remove debris from each hole, and then the expansion anchors were installed. The expansion anchors were checked to ensure that the base of the cone was flush with the threaded rod, and the assembly was tightened finger tight. The anchors were then placed in the holes cone side down. An anchor setting tool was placed through the threaded rod to drive the anchor into the pavement using a small sledgehammer. The anchor was finally driven into the pavement until the washer was flush with the pavement surface.

4. Remove damaged pavement section(s)

   Once expansion anchors were installed, the nut and washer from each installed expansion anchor was removed, and a swivel hoist lifting point and washer were inserted over the threaded anchor rod. The swivel hoist lifting points used in this investigation were 7.5-ton swivel lifting hoists manufactured by Crosby Group. The nut and washer were then returned to the anchor, and an impact wrench was used to tighten the nut of the anchor. A torque wrench was then used to tighten the nut until 250 ft-lb of torque was reached. Any excess threaded rod was then removed using a portable band saw. This step is shown in of Figure 52c.

   A crane was used to lift the 10-ft x 10-ft pavement section. For this project, 4 round slings were shackled together and installed on the hook of the crane. Each swivel lift point was connected with a sling. Care was taken to ensure that the nut and safety clip were installed prior to lifting. This step is shown in Figure 52d. Once the lifting points were connected to the crane, the 10-ft x 10-ft section of pavement was slowly lifted from the test section and removed from the site (Figure 52e).

5. Prepare dowel receptacles

   Once the pavement section was removed, a 22-lb electric jackhammer with 1.5-in. chisel was initially used to remove the pavement between the parallel saw cuts performed previously as shown in Figure 52f and Figure 52g. The jackhammer was inserted at a slight angle at the back of the painted boundary. Care was taken to ensure that the receptacle was fairly rectangular in shape with even dimensions across each side. Each receptacle was cleaned with compressed air, a steel brush, and a shop vacuum. The final prepared area is shown in Figure 52h.

   The use of a 22-lb jackhammer was labor intensive, and made it difficult to maintain operation tempo. The equipment had a tendency to become lodged during excavation. Dowel receptacle excavation typically required 5-7 minutes per dowel receptacle for each operator with the jackhammer shown in Figure 52g. Additionally, circumstances dictated excavating the dowel receptacles for several panels prior to removing the cut-out slab. The average dowel receptacle excavation in these conditions time averaged 8-10 minutes per dowel for each operator when the slab was not removed first.

   These times made it difficult to maintain the required repair tempo. Subsequent investigations showed that the average dowel receptacle excavation time could be reduced to less than a minute with a 35-lb-pneumatic jackhammer with 3-in. chisel (if the saw cut panel was removed).

Installation of bridge plate anchors and lifting points

The next step in the repair process was to install bridge plate anchors that connected the precast panel (P1) to the parent slabs during seating. Figure 53 shows the location of the bridge plates for all three repairs conducted. To save time, this step was conducted when the distressed area was being prepared. For single-panel repairs, 4 bridge plates were required. The bridge plates aided in installation of the precast panels by providing additional support during the seating of panels and prevented the panels from settling in the base material. An anchor template was used to mark the drilling locations for installing ½-in. concrete expansion anchors into the parent slabs to secure the bridge plate. A hammer drill with a 0.5-in. masonry drill bit was used to drill approximately 6-in. deep holes. The holes were then cleaned with compressed air. The concrete expansion anchors

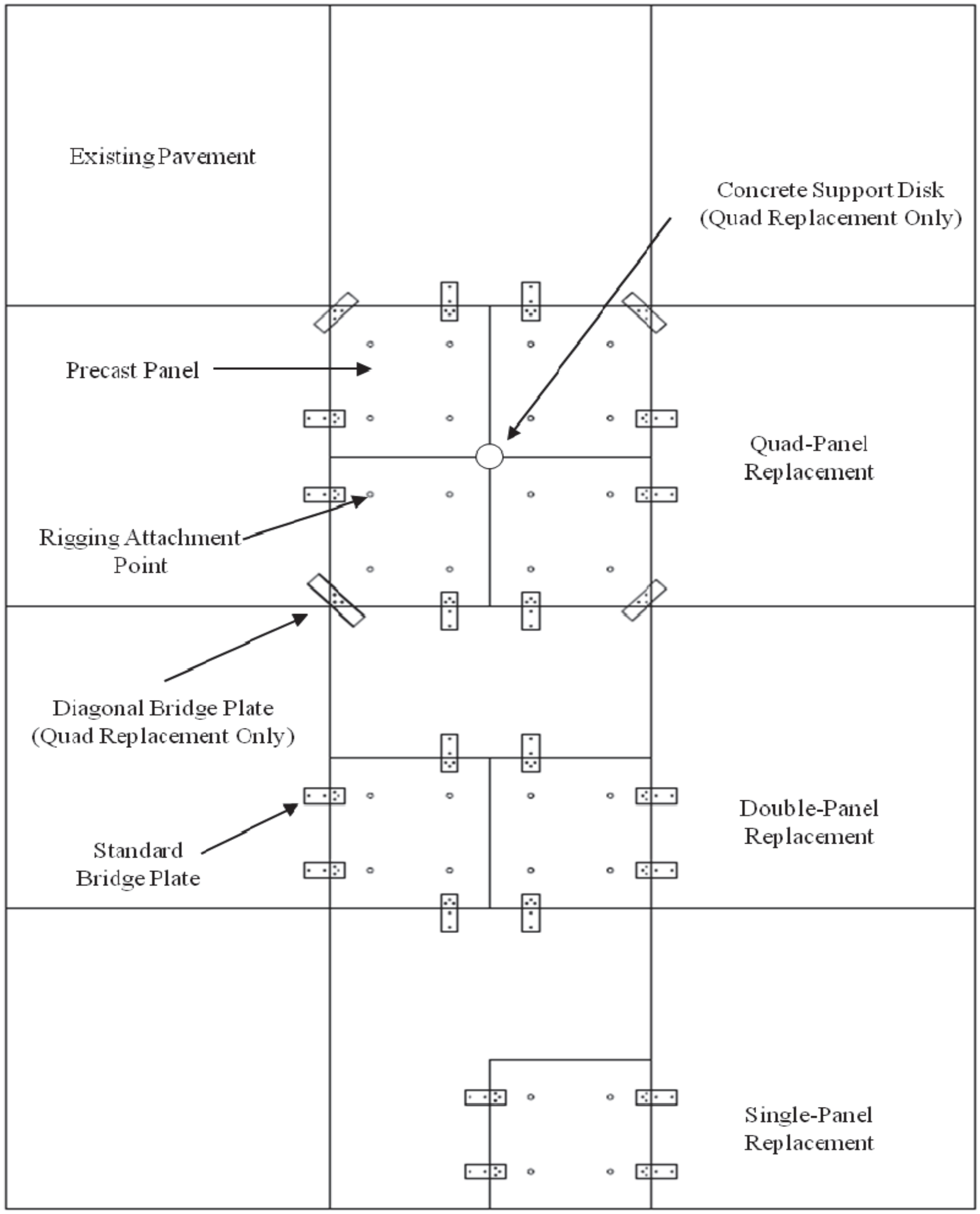

Figure 53. Bridge plate configurations for the repairs.

were inserted cone side down without the nut and washer and were driven into the parent slabs with a small sledgehammer (installation with the nut and anchor is not recommended as hammering the nut during installation will damage the threads of the rod and prevent tightening of the nut). The bridge plates were then placed over the installed concrete anchors to ensure correct alignment. The bridge plates were removed, and the washer and nuts were returned to the anchors and tightened until hand tight. The nuts were then tightened with a torque wrench until 55 ft-lb of torque were reached. This process is shown in Figure 54.

Figure 54. Installation of bridge plates: clockwise from top left: using template to mark anchor locations, drilling holes for anchors, installing anchor, and completed bridge plate installation and lifting eyes.

Base preparation

Immediately following the removal of the damaged slab, the next step in the process was to prepare the base layer. Because the parent slab was approximately 14-in. thick and the precast repair panels were only 11-in. thick, a new 3-in.-thick base course layer was needed for seating the precast panels. Flowable fill was selected for the base layer due to the self-leveling nature of the material and because of its acceptable performance in previous AFRL precast panel field tests. This material is a blend of portland cement, fly ash, fine mineral aggregate, and water. The flowable fill was designed and delivered from a local ready-mix plant in a standard transit truck. The flowable fill mix design is presented in Table 14.

Prior to placing the flowable fill, the sides of the repair area were marked around the perimeter of the hole 11 in. from the surface to provide a visual mark for the top of the base surface. The flowable fill was placed in the hole and was distributed using concrete rakes and flathead shovels. During placement, the flowable fill was extremely wet indicating that the w/c ratio was higher than expected. The vendor was contacted to make adjustments to the mix for the subsequent repairs.

Table 14. Flowable fill mixture design.

Materials:	Supplier	Spec	Test Method
Cement	Suwannee Cement Company	Type I/II	ASTM C-150
Fly Ash	STI-Pro Ash Company	Class F	ASTM C-618
Coarse Agg.	NA	Grade # 67	ASTM C-33
Fine Agg.	Sikes Sand Company	Silica Sand	FDOT 901
Admixture #1	Euclid Chemical Co., WR	Type A/D	ASTM C-494
Admixture #2	NA	Type F/G	ASTM C-494
Admixture	Euclid Chemical Co., AEA928	Type F/G	ASTM C-494

Material per cu. yd	SSD Weights	Specific Gravity	Absolute Volumes	
Cement, lbs	100	3.15	0.51	cubic ft
Fly Ash, lbs	500	2.44	3.28	cubic ft
Coarse Agg., lbs	0	2.6	0	cubic ft
Fine Agg., lbs	2260	2.63	13.77	cubic ft
Air Volume			4.05	cubic ft
Water, Gals.	40			
Water, lbs	333	1	5.34	cubic ft

Test Data	
Slump Range	NA
Air Range	15.00%
Unit Weight	118.47 lbs
W/C Ratio	0.56
Cememtitious	600 lbs

The flowable fill markings were overfilled by approximately 1/8 in. The material was then screeded using a specially designed metal screed that fit into a 10-ft-wide hole with a depth of approximately 11 in. (less the 1/8 in. overfill). Plywood was laid over the dowel receptacles to allow screeding; without the plywood, it was impossible to screed over the removed receptacle sections. The screed was used to level the surface of the flowable fill while still allowing adequate depth for the panel to be placed. During screeding any excess material was removed. The flowable fill was screeded twice using perpendicular screedings to ensure smoothness in both directions. Screeding is presented in Figure 55a.

Precast panel placement of Panel 1

Prior to the placement of the flowable fill, Panel 1 (P1), a standard panel (with attached bridge plates), was secured to the crane using rigging hardware connected to each of the pre-installed swift-lift attachment points and was positioned next to the repair location. Immediately following

Figure 55. Placement of precast panel: a) screeding of flowable fill; b) placing panel; c) placed panel with plywood spacers; d) close up of dowels in dowel receptacles; e) using compactor to seat panel; and f) panel prior to dowel receptacle grouting.

placement and leveling of the flowable fill as described in the previous section, the panel was placed using the crane (Figure 55b). Various thickness plywood spacing shims of thicknesses 1/8-, 1/4-, and 1/2- in. were placed in each corner of the prepared repair to maintain at least a 0.375-in.-wide joint between the precast panel and the existing slab and to protect the corners of the precast panel (Figure 55c). Care was taken to prevent misalignment of the dowels within the dowel receptacles. Proper alignment is shown and presented in Figure 55d. The installed panel did not completely compress the flowable fill layer, which had been slightly overfilled. A small vibratory roller compactor was rolled across the center of the panel to fully seat the panel (Figure 55e). A full-sized plywood sheet was placed prior to roller operation to prevent damage to the precast panel. The final placed panel is shown in Figure 55f.

Sealing the dowel sleeves and joints

The final installation procedure entailed filling the dowel receptacles and sealing the joints. A rapid-setting spall repair material, Pavemend 15.0™, was used to fill the dowel receptacles. It was mixed following manufacturer guidance using drills and paddles. This rapid-setting grout was selected for use based on previous AFRL field testing of dowel grouting for precast panels. Once mixed, each bucket of material was emptied into the dowels receptacles. The grout flowed outside the edge of the parent pavement and into the joints surrounding the panel. The material was self-leveling and did not require surface finishing. The joints between the PCC slab and the precast panel were filled using backer rod and silicone-based joint sealant. The placement of the grout is shown in Figure 56. Additionally, the final completed repair is shown in this figure. After at least 2 hrs of cure, the bridge plates were removed, and the anchors were cut and ground flush with the surface to reduce tire hazards.

Figure 56. Left: placement of rapid-setting grout in dowel receptacles; and right: grouted and sealed panel.

Repair 2- double-panel repair

Repair 2 was conducted during early November 2011 using a team of 12 people. The following sections detail the process and steps required for conducting a double-panel repair.

Distressed area removal

For this repair, two 10-ft x 10-ft sections of Slab 8 were removed. The process used to complete Repair 2 followed the steps detailed for Repair 1.

First, the repair boundary and the locations of the dowel sleeves were marked. Then, a saw was used to cut two 10-ft x 10-ft square repair areas along with all the dowel receptacle cuts. A total of 80 individual dowel receptacle cuts were required.

After performing the cutting procedures, concrete expansion anchors were installed in both concrete sections and swift-lift attachments were installed in one section, and the pavement was removed using the crane as detailed in the previous repair. The swift-lift attachments were then attached to the second pavement section, and that section was removed.

Once the pavement sections were removed, the dowel receptacles were prepared by removing the pavement between parallel saw cuts using a jackhammer and then cleaned as described for Repair 2.

Installation of bridge plate anchors and lifting points

Bridge plate anchors were then installed in both panels to aid in seating the panels. As with Repair 1, four bridge plates each were attached to Panel 2 and Panel 3 (P2 and P3). These panels were standard panels (with attached bridge plates), and bridge plates were installed in the same manner as Repair 1 for both panels.

Base preparation

Following the removal of the distressed areas, a new base course layer was constructed using flowable fill following the process detailed for Repair 1. The material consistency of the flowable fill was deemed better than the previous repair's (less wet). The flowable fill was used to prepare enough base material for a single panel to be placed. Following the placement of one panel, the base for the second panel was prepared. This allowed any excess flowable fill on one side of the repair to flow under the panel to the unfilled side to prevent any excess flowable fill from having to be removed from the dowel receptacles.

Precast panel placement of Panels 2 and 3

Prior to the placement of the flowable fill, both panels were positioned next to the repair location. As detailed in the previous section, immediately following placement of the flowable fill for one half the repair area (Figure 57a), the surface was leveled (Figure 57b), and Panel 3 was placed

Figure 57. Placement of two panels in Repair 2: a) placing flowable fill and reading first panel; b) screeding flowable fill for first panel; c) placing first panel; d) placed flowable fill for second panel; e) placing second panel; f) seating panel using roller compactor.

using a crane in a similar manner to Repair 1 (Figure 57c). Following the placement of the first panel, the flowable fill was prepared for the remaining repair area (Figure 57d), then Panel 2 was placed (Figure 57e). As with Repair 1, a roller compactor was used to seat the panels (Figure 57f). Any excess flowable fill that filled around the dowels was then removed.

Sealing the dowel receptacles and joints

As with Repair 1, the final installation procedure entailed filling the dowel receptacles and the joints following the same procedure detailed previously with the following exception. For the double- and quad-panel repairs, a

0.5-in.-wide construction joint was designed due to the panel and parent slab geometries. During placement of Panels 2 and 3, the installation team allowed for an excessive construction joint during installation of the first panel, along the longitudinal edge between the existing PCC and the precast panel. This action decreased the available joint space between the precast panels and between the second installed panel and the existing PCC. The longitudinal joint between the precast panels was less than 0.125 in. after the second panel was placed. The two panels made contact with one another, which resulted in minor spalling near the joint. This also necessitated using the walk-behind saw to saw cut an adequate longitudinal joint between the two installed precast panels. Following this remediation, the joints were sealed using backer rod and then joint sealant. Following at least 2 hrs of cure, the bridge plates were removed from the panels.

Repair 3- quad-panel repair

Repair 3 was conducted during early November 2011. The following sections detail the process and steps required for conducting a quad-panel repair.

Distressed area removal

For this repair, four 10-ft x 10-ft sections of Slab 11 were designated for removal. The process used to complete Repair 3 followed the steps detailed for the previous repairs with a few differences. As with the previous repairs, the repair boundary and the locations of the dowel receptacles were marked. Then a saw was used to saw cut four 10-ft x 10-ft square repair areas in Slab 11 along with 80 individual cuts for the 40 dowels in this repair.

After performing the cutting procedures, concrete expansion anchors were installed in all four concrete sections to be removed, and swift-lift attachments were installed in one section. Then the pavement was removed using the crane as detailed in the previous repair. This process was repeated until all four sections were removed. Once the pavement sections were removed, the dowel receptacles were prepared by removing the pavement between parallel saw cuts using a jackhammer and then cleaned as described for the previous two repairs.

Setting of the concrete pedestal

An additional step was required for the quad-panel repair. After the panels were removed, a 3-in.-thick concrete pedestal was installed in the center of the repair area to act as a support for the panel corners to prevent the inside corners from sinking into the flowable fill. The pedestal location is shown in Figure 58a. The pedestal was placed by setting perpendicular string lines to identify and mark the intersection of the four interior panels. A 2-ft. square x 1-in.-deep section of base course material, centered at the string line intersection, was excavated to allow the installation team to make fine-tuned adjustments to the pedestal elevation. A 1-in.-thick layer of concrete sand was placed in the excavated section, and the pedestal was placed on top of the concrete sand and twisted into place until the top of the pedestal was 11 in. below the string line.

Figure 58. Installation of Repair 3 panels: a) pedestal location; b) placing first panel; c) using shims to place second panel; d) placing flowable fill for third and fourth panels; e) placing third panel; f) placing fourth panel.

Installation of bridge plate anchors and lifting points

Bridge plate anchors were then installed in all four panels to aid in seating the panels. The placement of bridge plates varied from the previous two repairs. Bridge plates were installed in the outside corners of two sides of the panels, and a plate was installed across the outermost corner of each panel as shown in Figure 53. No bridge plates were installed on the inner side of the panels because there was no parent PCC onto which to rest the plate. The bridge plate set up is also shown in Figure 58.

Base preparation and installation of Panels 4-7

Following the removal of the distressed areas, a new base course layer was constructed using flowable fill. The flowable fill was used to prepare enough base material for two panels to be placed (Panels 4 and 6). Panel 4 was a terminal panel, and Panel 6 was a standard panel. Following the placement of Panel 4 (Figure 58b), Panel 6 was installed both using the crane in the same manner as described for the previous two repairs (Figure 58c). Care was taken when installing Panel 6 to align the dowels in the preformed dowel receptacles in Panel 4 and the dowel receptacles in the parent PCC. Following the placement of Panels 4 and 6, flowable fill was placed to provide base for installing the remaining panels (Figure 58d). Panel 7 was installed followed by Panel 5 (Figure 58e and 58f). During the seating of Panel 5, the inside edge of Panel 4 was broken. This area required repair during the placement of the dowel receptacle grout as shown in Figure 59. There were no issues seating Panels 4, 6, and 7, because excess flowable fill material was able to flow into unoccupied quadrants of the repair section. The excess material migrated into the Panel 5 repair section, as it was the last panel placed. This section was screeded prior to panel placement and an attempt was made to remove excess material. However, when Panel 5 was placed there was still excess flowable fill material and the panel was seated approximately ½ in. higher than the existing PCC and the previously installed precast slabs. A vibratory roller was utilized during the single- and double-panel repairs to fully seat the panels. The quad-panel repair bridge plate configuration was different than the previous repairs and did not allow adequate space for the vibratory roller to drive onto the panels. A smaller, portable vibratory roller was placed on the panels. This roller was only partially effective, and Panel 5 was ultimately seated approximately 0.25 in. higher than the existing PCC and precast panels.

Figure 59. Left: broken corner of a panel, and Right: repair of broken corner.

Sealing the dowel sleeves and joints

As with previous repairs, the final installation procedure entailed filling the dowel receptacles and the joints following the same procedure detailed previously. Unlike the previous repairs, additional dowel receptacles in the terminal panels also had to be sealed. In this step in the process, all dowel receptacles were filled with rapid-setting repair material, and the broken corner of Panel 4. Sealant was placed in the joint between Panels 4 and 5 and 6 and 7 using backer rod and then joint sealant.

Lessons learned during panel installation

Several items were noted during the placement of the three repairs. Lessons learned are presented in this section.

- All materials and equipment must be ready and on site prior to repair actions. The crane should be near the repair area with all precast panels within attachment range to minimize repair time and the handling of panels. Minimizing handling of panels will prevent damage during transport.
- Preparation of the dowel receptacles was one of the most time consuming steps in the process due to the need to place numerous small cuts for the dowels to be tied to the existing pavement. The damaged pavement should be first removed prior to preparing the dowel receptacles. The use of a 22-lb jackhammer was determined to be too labor-intensive and slow to meet rapid repair timelines. The use of a 42-lb jackhammer with a 3-in. chisel bit was identified as the optimal size equipment for this task. Time to prepare these receptacles could be reduced if two saws and two jackhammers were used simultaneously during the repair process.

- The strength of the flowable fill combined with the bridge plates prevented the panels from sinking into the prepared base. Compactive action was required to seat the panels that were level with the surrounding pavement. The small vibratory compactor worked well for this purpose, but the compactor used must fit between the bridge plates on the panel requiring a small drum size. For the quad-panel repair, a small portable compactor had to be used. This compactor did not work as well as the larger compactor.
- The flowable fill placed in Repair 1 was very wet, and the material vendor was contacted to modify the mixture for the remaining repairs to allow easier placing. The mixture varied substantially in flowability between repairs. The consistency (visually) of Repair 2's flowable was the best of the three repairs with Repair 1's being the wettest.
- The plywood placed over the dowel receptacles was important to allow a smooth, uninterrupted surface for sliding the screed to level the flowable fill.
- The concrete pedestal worked well to support the corners of the panels in the quad-panel repair.
- The placement of the panels into the repair can be difficult. Ensuring proper alignment of the dowels in the dowel receptacles and maintaining a reasonably equal spacing around the panels is important to ensure good performance during trafficking. The crane operator may need to raise and lower the panels several times with at least 3 people working to place the panel properly using shims to maintain a consistent joint around the repair.
- The plywood shims work well to ensure a proper construction joint. Care should be taken to prevent excessive construction joints during panel installation, particularly connected panel installations. Using the saw to saw cut an adequate longitudinal joint between the two installed precast panels worked well to remediate the poor spacing resulting from installation.
- Achieving the proper flowable fill elevation to seat panels is a challenging aspect of the repair operation. Too little material can result in installed panels that are seated below the existing PCC (if bridge plates are not used), or an installed panel that may have voids beneath it (if bridge plates are used). Excess material can result in installed panels seated above the existing PCC. This typically requires use of a vibratory roller to fully seat the panels. The optimal method appears to be to overfill the flowable fill layer by approximately 0.125 in. This

slight overfill acts to eliminate voids and reduce installation times by reducing excess material.

- During the placement of one of the panels in the quad-panel repair, one of the corners of the panel damaged. The spacing of the outer dowels in the panels should be redesigned to reduce the likelihood of breaking a small piece of concrete from the corner (less than 6 in. wide). A distance of at least 1 ft from the edge is recommended for future designs.
- Generating the volume of rapid-setting grout used to fill the dowel receptacles using repair materials packaged in 5-gal buckets was labor and time intensive. Alternative materials that can be mixed in bulk should be included.
- Although 12 people were used for this effort, a team of 8 may be more appropriate. Not all personnel were engaged at all times resulting in congestion at the site. The team must include a crane operator that remains in the crane for most of the repair effort.

A picture showing the three repairs after installation is provided in Figure 60.

Figure 60. Completed Repairs 1-3.

Installation timeline

This section summarizes the installation process timeline for the three precast panel repair operations. The timeline data were compiled during the precast concrete panel installation phase of this project using video footage and notes taken during the events. For rapid installation of panels, all equipment, personnel, and materials must be readily available on site prior to beginning installation. The time to move and set up for panel installation was not recorded, nor are these data included in the installation timeline. Additionally, while a task was being completed, materials and equipment were readied for the next task(s). This ensured the most rapid placement of the panels.

A minimum cure time of 2 hrs was recommended for the dowel receptacle filling material to reach a minimum of 3,000 lb/in.^{2} unconfined compressive strength and the flowable fill to reach the required compressive strength based on projected aircraft (55 lb/in.^{2} for the F-15E and 85 lb/in.^{2} for the C-17). The time to remove the bridge plates was included in the curing time of the dowel receptacle fill and flowable fill. During this curing, cleanup and removal of equipment from the site was also conducted.

Following review of the installation timelines, recommendations were made to optimize the repair installation process by including simultaneous activities including saw cutting of distressed areas and dowel receptacles, installation of lifting anchors, dowel sleeve excavations, and dowel and joint sealing efforts. Current and optimized timing for the installation process is included in Table 15.

Table 15. Current and proposed timing for completing repairs.

	Current Timing, min.			Optimized Timing, min.		
	Repair Type			Time (minutes)		
Task	Single-panel	Double-panel	Quad-panel	Single-panel	Double-panel	Quad-panel
Mark perimeter of distressed slab	10	15	20	10	15	20
Saw cutting operations	15	35	60	7.5	17.5	30
Dowel receptacle cutting	25	50	50	12.5	25	25
Anchor drilling	15	30	60	5	10	20
Anchor installation	5	10	20	5	10	20
Attach crane rigging hardware	1	2	4	1	2	4
Lift distressed section	5	10	20	5	10	20
Dowel sleeve excavation (existing PCC slab)	30	60	60	15	30	30

Task	Current Timing, min.			Optimized Timing, min.		
	Repair Type			Time (minutes)		
	Single-panel	Double-panel	Quad-panel	Single-panel	Double-panel	Quad-panel
Flowable fill placement	10	20	25	10	20	25
Precast panel placement	5	10	30	5	10	30
Compaction (if needed)	5	10	20	5	10	20
Removal of flowable fill from dowel receptacles	3	6	10	3	6	10
Placement of joint and dowel sealant	30	60	90	15	30	45
Dowel receptacle finishing	5	10	20	5	10	20
Curing	120	120	120	120	120	120
Total repair time, minutes	284	448	609	224	325.5	439
Total repair time, hr	4.73	7.47	10.15	3.73	5.43	7.32

Note: Optimized timing assumes doubling the manpower and equipment to perform saw cutting, dowel receptacle cutting, expansion anchor installation, and joint and dowel sealing efforts.

As can be seen in the table, none of the panel repairs could be conducted in less than 4 hrs during the field testing. Through modification to the repair technique, the single-panel repair could possibly be conducted in less than 4 hrs. Through optimization, the single-panel repair timing could be reduced by 1 hr, the double-panel repair by slightly over 2 hrs, and the quad repair by almost 3 hrs. If the repairs are to be conducted in 4 to 6 hrs, then only the single- and double-panel repairs could meet this objective using the optimized repair approach. In the current form, none of the panel repairs would be applicable for emergency repairs. The quad-panel repair requires close to 10 hrs to complete this size repair, outside the objective timeframe of 4-6 hrs. The timing of repairs using the optimized method is recommended to determine if the estimated time savings can be obtained.

A potential solution to the timing issue is to divide the repair tasks into two separate repair periods (i.e., do a portion of the repair one night or during low traffic periods and complete the repair during the next available repair window). During the first repair period, the tasks of marking, saw cutting, and drilling locations for the lifting anchors could take place, which would still allow aircraft operations when this repair period concludes. During the second repair period, the remaining repair tasks would be completed including curing.

Table 16 presents the time for each phase of repair. As can be seen in the table, splitting the repair tasks into two phases reduces the operational downtime to conduct repair operations. The second phase of the repair process is the more time consuming phase due to curing of the dowel receptacle material. Despite this cure time, the second phase of repair for a quad panel is less than 6 hrs using the optimized repair procedure or 7 hrs with the current procedure. Similar reductions in time are shown for the double- and single-panel repairs, but the biggest impact is seen for the quad panel that cannot be completed using either the current or optimized procedure in less than 6 hrs. If 7 to 11 hrs are not available in a single repair window to complete this size repair, then this may be a viable option.

Table 16. Current and proposed timing for completing repairs during two separate repair phases.

Task	Current Timing, min.			Optimized Timing, min.		
	Repair Type			Repair Type		
	Single-panel	Double-panel	Quad-panel	Single-panel	Double-panel	Quad-panel
Phase 1						
Mark perimeter of distressed slab	10	15	20	10	15	20
Saw cutting operations	15	35	60	7.5	17.5	30
Dowel receptacle cutting	25	50	50	12.5	25	25
Anchor drilling	15	30	60	5	10	20
Total phase I repair time, min.	*65*	*130*	*190*	*35*	*67.5*	*95*
Phase 2						
Anchor installation	5	10	20	5	10	20
Attach crane rigging hardware	1	2	4	1	2	4
Lift distressed section	5	10	20	5	10	20
Dowel sleeve excavation (existing PCC slab)	30	60	60	15	30	30
Flowable fill placement	10	20	25	10	20	25
Precast panel placement	5	10	30	5	10	30
Compaction (if needed)	5	10	20	5	10	20
Removal of flowable fill from dowel receptacles	3	6	10	3	6	10
Placement of joint and dowel sealant	30	60	90	15	30	45
Dowel receptacle finishing	5	10	20	5	10	20
Curing	120	120	120	120	120	120
Total phase II repair time, min.	*219*	*318*	*419*	*189*	*258*	*344*

Note: Optimized timing assumes doubling the manpower and equipment to perform saw cutting, dowel receptacle cutting, expansion anchor installation, and joint and dowel sealing efforts.

Development of a deployable containerized kit

Following the fabrication and placement of panels, a listing of the disposable construction materials and small equipment items needed to produce 12 precast panels were generated for assembling a deployable containerized kit. This list was based on input from engineers, technicians, and laborers during fabrication and placement of the panels. Items not included in the kit included heavy equipment such as a crane, concrete mixer, truck, or vibratory compactor. No constituent materials were provided to make either the PCC or flowable fill. These items must be secured by the base anticipating precast panel repairs.

This list of equipment and material will be finalized following the trafficking of the panels in the phase II portion of this report; thus, the items in this list may change as a result of performance issues encountered during trafficking. The draft listings of materials and equipment are provided in Appendix C.

7 Conclusions and Recommendations

A number of investigations have been conducted over the last 50-80 years into using precast PCC panels for pavement repair and construction. A review of the literature led to the selection of the Air Force precast panel repair method using a small panel over flowable fill. The Air Force's panel design was reviewed and modified to ensure the panel could be used alone or in series with other panels to conduct partial- and full-slab repairs on airfield PCC pavements. The design of the panels, fabrication methods, and field panel installations are presented in this report. A preliminary listing of equipment and materials to produce a deployable precast panel repair kit was also provided. Based on the preparation of the panels and the field installations, the following conclusions and recommendations are made:

Lifting, handling, and storage:

- The minimum required lifting capability on an airbase for placing precast panels is 15 tons. A 30-ton capacity crane is recommended for a quad-panel installation.
- Panels should not be stored more than 4 panels high, and dunnage must be properly oriented under the lifting points to prevent damage of the panels during storage.
- Adding the additional upper layer of #5 reinforcement is recommended to allow for stacking the panels in both the longitudinal and transverse directions. Reinforcement optimization should be considered to conform to ACI design standards and/or minimize material quantities.
- Minimize panel handling prior to installation to prevent damage during transport.
- At present, the use of a forklift is not recommended for placing the precast panels. It is recommended that the design of an attachment for a forklift be explored to allow the use of a forklift in the future." delete what is currently there after the word panels.
- It is recommended that the design of an attachment for a forklift be explored to allow the use of a forklift for placing the panels.

Material requirements:

- An adequate preparation and storage area with overhead clearance and a hardened pavement surface is preferred to fabricate the panels.

- PCC and flowable fill must be available to conduct these repairs. The minimum 28-day compressive strength for the panel PCC is 5,000 lb/in.^{2} corresponding to 650 lb/in.^{2} flexural strength.
- The minimum flowable fill compressive strength at 2 hr is 55 lb/in.^{2} for F-15E traffic and 85 lb/in.^{2} for C-17 traffic. These strength values include a safety factor of 2.
- In addition to the PCC and flowable fill, a rapid-setting grout must be available to fill the dowel receptacles. The material should reach at least 3,000 lb/in.^{2} compressive strength after only 2 hr of cure.

Dowel receptacle preparation and sealing:

- Batching larger volumes of rapid-setting grout at a time is recommended to reduce the time required to complete the repair. Proper selection of materials that allow for larger batch sizes will be critical to success with this method. The damaged pavement should be first removed prior to preparing the dowel receptacles. The use of a 22-lb jackhammer was deemed too labor-intensive and slow. A 42-lb jackhammer should be used for this task.
- It is recommended that a rapid-setting grout that can be mixed in larger quantities be used to reduce the time required to complete the repair.
- The grout must be placed carefully to prevent overfilling the repairs and causing buildup of material outside the dowel receptacles.

Panel leveling and seating:

- The strength of the flowable fill combined with the bridge plates prevented the panels from sinking into the prepared base. A small compactor worked well to seat the panels, but the compactor used must fit between the bridge plates on the panel. A small, portable compactor did not work as well as the larger compactor.
- The plywood over the dowel receptacles was required to allow a smooth, uninterrupted surface for screeding the flowable fill.
- The concrete pedestal worked well to support the corners of the panels in the quad-panel repair.
- Ensuring proper alignment of the dowels in the dowel receptacles and maintaining a reasonably equal spacing around the panels and the parent PCC is important to ensuring good performance during trafficking.

- The width of the construction joint should be maintained as close to the design widths as possible to prevent spalling of the panels when placed too close together or loss of load transfer efficiency if spaced too far apart.

Personnel:

- A team of 8 including a crane operator is required to perform the repairs.
- To conduct repairs as quickly as possible, each subsequent repair step must be conducted immediately following the completion of a task. Planning and preparation of techniques, tactics, and procedures are required to ensure repairs can be completed in the required timeframes.

Speed:

- Additional repairs are recommended to determine if the estimated time savings can be obtained using the optimized repair method.
- Dividing the repair tasks into two separate phases will potentially allow the quad-panel repair to be completed in two shorter repair windows each less than 8 hrs.

Precast kit:

- If the panels perform well under traffic tests, a deployable kit should be developed to include all small equipment required to fabricated panels and complete precast panel repairs. Integration of the kit items required for work tasks into currently developed or under development repair systems may be a more cost effective measure for implementation. Heavy equipment and disposable materials used i large quantities would have to be obtained locally.
- It is recommended that the equipment and materials in the deployable kit be used to conduct timed precast panel repairs using a military repair team to identify any additional material needs or installation/fabrication process modifications.
- The modified formwork package is recommended for use in a deployable kit. The original panel construction steps will require modification from that depicted if the new formwork is used.
- The performance of the precast panels will be determined through full-scale field testing. Based on field testing and user defined needs, the list of materials should be modified and the kit packaged for use.

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Appendix A: Panel Fabrication Instructions

Preparing for Panel Fabrication

1. Determine the type and quantity of panels to be made as shown in Figure A1.

   - Standard: Male-Male: exposed dowels on 2 opposite ends of panel
   - Terminal: Male-Female: exposed dowels on 1 end and dowel receptacles on the opposite end

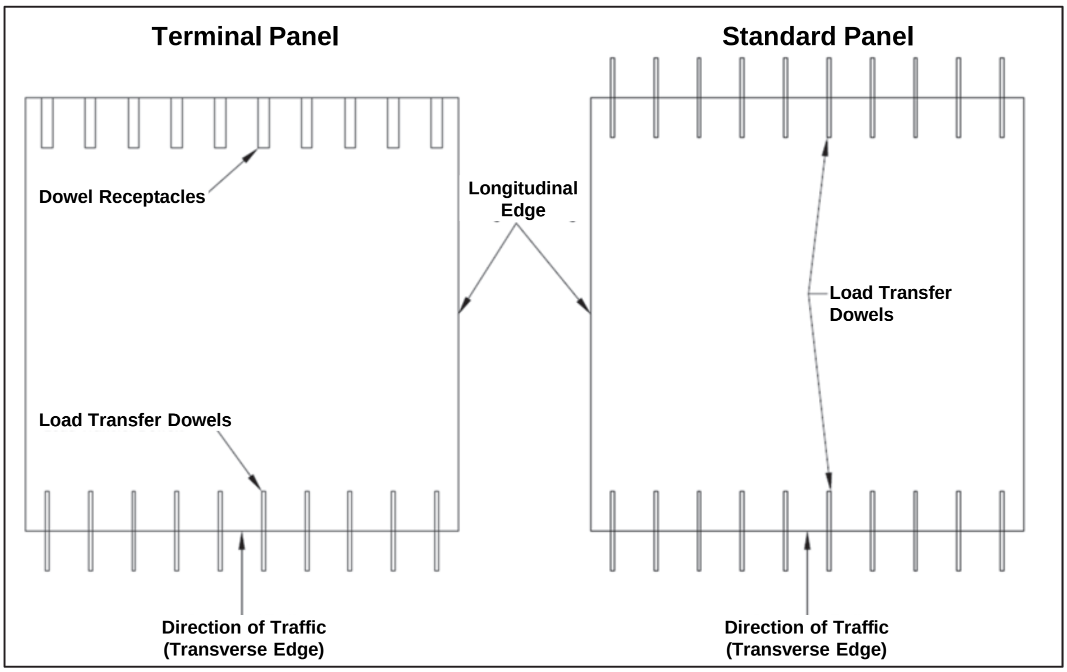

Figure A1. Panel types.

2. Find a suitable construction area with smooth level surface.

   - Required:

     - Paved surface such as concrete or asphalt
     - Installed or portable electrical source
     - Installed or transportable water source

   - Recommended:

     - Large enough to construct all panels without removing panels immediately

- Semi- or completely enclosed building providing sufficient overhead height to operate crane

3. Find and prepare suitable storage area.

    - Obtain proper dunnage to stack panels for easy retrieval. Two pieces of 4-in.x4-in.x10-ft pressure treated lumber are required for each panel.
    - Store dunnage outside.

        - Do not begin stacking panels on dunnage until 28 day strength is reached or a 5,000 lb/in.^{2} can be confirmed.
        - Place each piece 30 ± 6 in. from each slab edge, parallel to dowels (longitudinal direction).
        - **Do not stack panels more than 4 tall.**

4. Move all materials required for construction of the panels to the chosen construction site.
5. Remove/sweep debris from paved construction surface.
6. Order concrete.

    - Each panel requires 4 yd^3 of concrete in volume after 15 percent waste.
    - Order 8 yd^3 of concrete for casting 2 panels at a time.
    - Specifications:

        - 5,000 lb/in.^{2} compressive strength
        - 0.75 in. nominal maximum aggregate size
        - 6 ± 1 percent air by volume is required to prevent freeze-thaw damage from outdoor storage
        - 4 ± 1 in. slump

7. Organize and connect formwork (Figure A2). (10 min total, 2 people)

    - There is enough formwork in the deployable kit to construct two slabs of either configuration type at a time. Bolt smaller side form pieces together to make sides with 10-ft nominal lengths. Components of the formwork can weigh from 100 -150 lb; care should be taken when handling pieces and completed formwork lengths.
    - Find the etched labeling system at the middle of each piece of formwork. Place connecting lengths beside one another.

Figure A2. Organized formwork.

- Undoweled sides will be of equal length. Doweled sides are cut 6 in. off center to prevent having the connection at the joint.
- Each complete form will consist of parts with the following ends:
    - Steel angle and a flat end with 2 thread bars (Piece 1)
    - Two flat ends, one with 2 large tapped holes and the other with 3 smaller tapped holes (Piece 2)

o Connect the form components together.

- Place Piece 1 next to Piece 2.
    - The flat end with 2 thread bars of Piece 1 should be facing the flat end with 2 large tapped holes of Piece 2.
- Begin turning the two nuts on the angle end of Piece 1. Align the two pieces of formwork such that the thread bars draw the two pieces together.
- Use an impact hammer to tighten and secure the connection slightly past finger tight. Do not drive the nut for more than 3 seconds to prevent damaging the formwork.
- Repeat on all other lengths needed.

8. Position, construct and prepare formwork (Figure A3). (60 min total, 2 people)

o Position form on the paved surface at the desired location.

- A minimum of 10 feet should be provided in between each slab to provide enough clear working space and allow for the future removal of forms.

o Match the etched labels at the corners (labeled “a” in Figure A3).

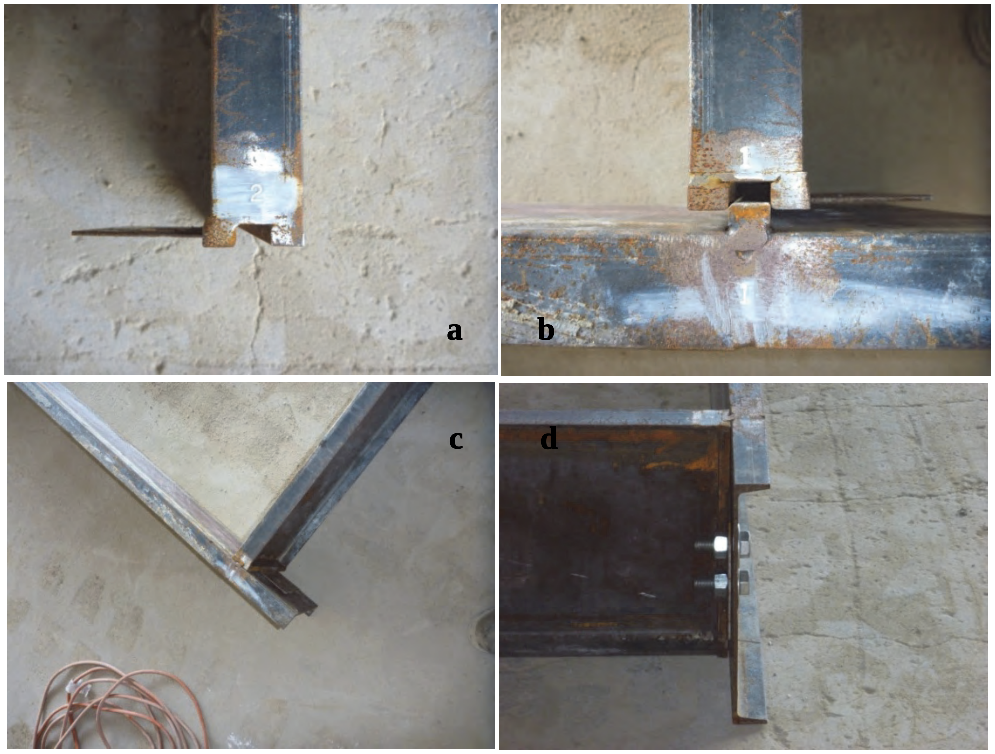

Figure A3. Connecting the formwork.

- The lengths of formwork connect perpendicular to each other at the angle from one length to the flat end of another (“b” and “c” in Figure A3).

o Place the bolts through the holes of the angled piece. Align the lengths of formwork so the bolts go through the tapped holes. Tighten with the impact hammer slightly past finger tight. (“d” in Figure A3)

- Three bolts are used to connect the lengths together.
- Use an impact hammer to tighten and secure the connection slightly past finger tight. **Do not drive the nut for more than 3 seconds to prevent damaging the formwork.**
- Ensure top of forms at each corner are flush with one another.

o Repeat on all other lengths needed to complete the square and check for squareness.
o Anchor the formwork to the pavement (Figure A4).

- Locate the concrete anchor tabs, centered on the bottom edge of each segment of formwork.

Figure A4. Anchoring the formwork.

- Use a 0.5-in. masonry drill bit and drill 6 in. into the pavement with the hammer drill (“a” in Figure A4).

    - Maintain the threaded rod within 5° of vertical.

- Do not attempt to widen the hole to make anchor installation easier. This will cause the anchor not to activate and hold the concrete. Install the first concrete anchor through the anchor tab. Drive the anchor into hole with the steel mallet. Use the ratchet or impact hammer to activate the anchor into the pavement (“b-d” in Figure A4).
- Remeasure and verify the diagonals of the formwork.

    - If the form is still squared, install a second concrete anchor in the opposite side.
    - If not squared, do not remove the anchor bolt, but gently nudge the corners until the diagonal measurements represent the form work has been re-squared.

- Continue to install anchor bolts on the remaining sides, checking the formwork squareness each time.

- Clean any excess previous mortar from surface edges of the forms with a square-point shovel, steel brush, scraper, etc. Use a mallet as needed to remove stubborn or thick deposits.
- Install the dowel bar holders (male panel length only).
    - Locate the dowel bar holders.
    - Insert the tube end through the formwork from the outside face.
    - Bolt the dowel holder into place using the two horizontal tapped holes. Use an impact hammer to tighten the bolts slightly past finger tight. Do not drive the nut for more than 3 seconds to prevent damaging the formwork.
- Install the dowel blockout. [female panel length only] (30 min total, 2 people)
    - Lubricate all threaded holes and bolts with grease to ensure concrete does not flow around threads.
    - Generously apply form release oil to all surfaces of the blockouts prior to installing to formwork.
        - Do not apply after installation to ensure release oil does not affect reinforcement embedment/bond to the concrete.
    - Install all blockouts across the side of the form.
        - Match holes of steel block out with holes in the formwork.
            - The top and bottom widths of the blockout are slightly different to allow for easy removal.
            - **The top surface is wider than the bottom and will be stamped "TOP". If the blockouts are installed incorrectly it will be extremely difficult to remove them from the finished pad and there is a serious damage potential to the precast slab.**
        - Install bolts through the holes to attach the blockout.
        - Ensure the top of each blockout is parallel and level to the top of the form
        - Tighten all blockout bolts with an impact wrench. Use an impact hammer to tighten the bolt slightly past finger

tight. Do not drive the nut for more than 3 seconds to prevent damaging the formwork.

- Generously spray the forms with form release agent. Spray all interior and exterior exposed surfaces to facilitate clean up and prevent concrete build up as shown in Figure A5

Figure A5. Application of release agent.

9. Construct the reinforcement grid (Precut: 30 min total, 2 people; Stock length: 60 min total if all reinforcement for all panels planned for construction are cut at once, 2 people).

    - Prepare the reinforcement.

        - If the rebar segments arrived precut, measure several pieces to ensure the correct length [9 ft 7.5 in. ± 0.5 in.].

            - If long, trim to correct size.
            - If significantly short [6 in. or more], discard and use another.

        - If the rebar was not precut, it will need to be cut to the correct length on site.

            - Measure and mark the cut location with paint

                - For each slab, the follow number of pieces are needed

                    - 18 - #3 bars at 9 ft 7.5 in. ± 0.5 in.
                    - 16 - #5 bars at 9 ft 7.5 in. ± 0.5 in.
                    - 6 - #3 bars at 1 ft ± 0.25 in. are required per slab constructed

            - Use a rebar cutter provided in the kit to cut the rebar to size

- Mark the intersections for each bar at the correct spacing

    - Use a tape measure and paint markings on the reinforcement. Markings can be made at one time if laid on a flat surface and ends are squared.

        - Place the first marking 9.875 in. from the bar end
        - Continue marking the rest of the bar at 12 in. intervals

- Construct the #3 reinforcement grid. (20 min each grid, 2 people)

    - Collect the 18 pieces of longer prepared #3 bars
    - Lay 9 bars on the ground to form the bottom layer of grid. Space the bars approximately 12 in. apart on centers as shown in “a” in Figure A6.
    - Lay the remaining 9 bars perpendicular to the previous bars. (“b” in Figure A6) Align the reinforcement so the paint marks overlap. Adjust any bars to best fit all the markings if the paint marks do not line up. The ends of the bars should be fairly flush with one another.
    - Tie the rebar at the intersections (“c”-“e” in Figure A6).

        - Loop a 6” precut tie under the bottom bar and bend it so the end loops overlap

            - Precut ties are easier to work with and minimize disturbing positioned bars
            - Traditional linesman pliers and 16 gauge steel tie wire can be used as well.

        - Place the hook of the ratcheting tying tool through the overlapped loops
        - Pull the rebar tool up to twist the tie. About 3 pulls should tighten and secure a 6-in.-long tie tightly.
        - Tie all exterior connections. Checkerboard the interior connections. Turn the looped ends of the completed tie down.

- Install the lower #5 reinforcement bars to the #3 rebar grid. (10 min each grid, 2 people)

Figure A6. Constructing the rebar grid.

- Mark the installation locations on the prepared reinforcement grid.
    - Place a paint marking at 2.5 in. from the bar end on the most exterior upper grid bars.
    - Make an additional set marks to the interior marking made at 5 in. on center.
- Collect 4 pieces of cut # 5 bar and lay them on the prepared grid. Align bars with the paint markings. Align the reinforcement to overlap the paint marks; however, adjust any bars to best fit all the markings if the paint marks do not line up. Ensure the ends

of the #5 bars are flush with one another and the underlying #3 bars.

- #5 reinforcement will be perpendicular to the upper level of the #3 reinforcement and parallel to the lower level of the #3 reinforcement.

- Tie the #5 bars to the upper layer of #3 bars as described earlier.

10. Install the reinforcement grid inside the formwork as shown in "f" in Figure A6. (15 minutes total, 4 people)

   - Lift and place the completed reinforcement grid within the constructed formwork.
      - Use safe lifting procedures. The constructed reinforcement grid weights 120 lbs total.
      - The #5 bars should be perpendicular to the form edges with dowel holes.
   - Ensure the grid is centered inside the formwork. There should be a 1.5 in. gap between the ends of the reinforcement and the formwork. Reposition as needed for best fit.
   - Place the 1.5 in. bar chairs underneath the lower #3 bar layer of the grid to hold into correct elevation (Figure A7).
      - Evenly space 25 chairs out to minimize any sagging and provide sufficient stability
      - Verify the grid is still centered within the formwork once completed.

Figure A7. Installation of chairs.

11. Install precut dowel bars. (45 min total, 2 people)

    - Mark dowel installation depth.

        - Find the center of each dowel [10 in.] to be installed and mark with a permanent marker.
        - Place a 1 in. x 5 in. strip of duct tape at the center of the dowel. The edge of the duct tape should touch the centerline of the dowel. The tape will help seal the void around the dowel receptacle.
        - Make a mark on the dowel from the taped end. This mark will indicate the proper insertion depth when used in the dowel bar holder.

            - From the side of the form, the dowel should stick out 10 in. – width of formwork – thickness of dowel holder system ± 0.125 in.
            - Example: width of web = 0.25 in., dowel holder plates on formwork = 1 in. → 10- 0.25 - 1 = 8.75 ± 0.125 in.

    - Lightly grease the end of the 10 in. side of the dowel.

        - Greasing the dowel allows for easier removal in case it's removal is necessary when removing formwork.
        - Ensure a light coat is used. Too much grease will leave voids around the dowels and cause future performance losses.

    - Install the greased dowel from inside the formwork by sliding the non-greased end from the interior of the formwork through the dowel hole as shown in Figure A8. This eliminates the loss of grease while sliding through the dowel receptacle.
    - Pack the dowel receptacle on the form with grease to keep concrete from flowing out of the gap between the dowel and the edge of the dowel receptacle.
    - Check the grease coating around the dowel for damage and repair as necessary. Remove any excess as needed.

12. Install the upper #5 reinforcement bar. (15 min per panel, 2 people). The upper layer of #5 reinforcement is in both the transverse and longitudinal directions.

Figure A8. Installation of dowels.

- There are four groups of three#5 bars that must be installed on each panel type. Their installation location depends on whether the form is designed to fabricate a terminal or standard panel.
  - Standard Panels. There are 12 total pieces of #5 rebar- six in the transverse direction(perpendicular to dowels) and six in the longitudinal direction (parallel to load transfer dowels).
    - Transverse upper layer of #5 rebar
      - A total of six upper layers of transverse pieces of #5 rebar will be installed; 3 at each doweled end.
      - Two of the bars at each end rest on top of the installed dowels, located at 4 and 9 in. on center from the transverse edge of the formwork (vertically above and parallel to the #5 bar installed on reinforcement grid). Mark these locations on the exterior most dowels by damaging the dowel bar's grease coat with your finger.
      - A third bar lies 14.5 in. on center from the edge of the formwork. Measure this location perpendicular to the exterior most dowel bars and place 6-in.-high chairs. Place an additional 6-in.-high chair centered between the two chairs previously placed to support the bar.
      - Place #5 bars at each of the locations marked. Tie the reinforcement to the dowel bar or chair it rests upon.
        - ➢ Use 8-in.-long ties when tying the rebar to the dowel.

- This group of #5 reinforcement will be parallel to lower layer of #5 reinforcement previously installed to the grid as shown in Figure A9.

Figure A9. Locations of #5 rebar.

- Longitudinal upper layer of #5 rebar
  - These two groups of #5 dowels will rest on top of and be oriented perpendicular to the previously installed upper layer of #5 rebar.
  - Mark the 1st and 6th pieces of the previously installed upper layer of #5 rebar at locations 4 in., 9 in., and 14.5 in. from the longitudinal edge of the formwork.
  - Place #5 bars at each of the locations marked.
  - Use 6-in. rebar ties to tie the intersecting rebar pieces together.

- Terminal Panels. These panels also have four groups of three #5 bars that must be installed.
  - Transverse #5 Rebar
    - Doweled end
      - Three transverse pieces of #5 rebar will be installed on the doweled end of the panel.
      - Two of the bars rest on top of the installed dowels, located at 4 and 9 in. on center from the transverse edge of the formwork (vertically above and parallel to the #5 bar

installed on reinforcement grid). Mark these locations on the exterior most dowels by damaging the dowel bar's grease coat with your finger.

- A third bar lies 14.5 in. on center from the edge of the formwork. Measure this location perpendicular to the exterior most dowel bars and place 6-in.-high chairs. Place an additional 6-in.-high chair centered between the two chairs previously placed to support the bar.
- Place #5 bars at each of the locations marked. Tie the reinforcement to the dowel bar or chair it rests on.
- Use 8-in.-long ties when tying the rebar to the dowel.
- This group of #5 reinforcement will be parallel to lower layer of #5 reinforcement previously installed to the grid as shown in Figure A9.

- Receptacle End (Female End)

  - Three transverse pieces of #5 rebar will be installed on the non-doweled, female end of the panel.
  - These #5 rebar pieces will be centered 14 in., 19 in., and 24 in. from the transverse female end of the form work. This rebar layout is designed to provide adequate support without impacting dowel receptacle fabrication at the female end. These rebar are also perpendicular to the load transfer dowels (on the other end).
  - Use three 6-in.-high rebar chairs to support each of the three #5 rebar pieces. Place chairs such that each end of rebar, as well as the center, is supported.
  - Use 6-in.-long rebar ties to tie rebar pieces to rebar chairs.

- Longitudinal #5 Rebar
    - These two groups of #5 dowels will rest on top of and be oriented perpendicular to the previously installed upper layer of #5 rebar.
    - Mark the 1st and 6th pieces of the previously installed upper layer of #5 rebar at locations 4 in., 9 in., and 14.5 in. from the longitudinal edge of the formwork.
    - Place #5 bars at each of the locations marked.
    - Use 6-in.-long rebar ties to tie the intersecting rebar pieces together.

13. Prepare the swift lift anchors (10 min total, 2 people)
    - Place the rubber recess former around the top end (thicker) of the anchor as shown in Figure A10.
    - Use duct tape to completely seal the seams and surface of the rubber recess former, to prevent mortar from entering during concrete placement. Minimize folds in tape used.

Figure A10. Swift lift anchor and rubber recess.

Concrete Placement and Form Removal

1. Place, consolidate, screed and finish concrete using general concrete practices. (45 min per set of 2 slabs, 4 people)
    - o Follow UFC 3-250-04FA for guidance on concrete placement.
    - o Use spud vibrator (1.5 in minimum head diameter) to consolidate concrete.
        - Move in and out vertically only. Space the vibrator insertions at 12 in. on center.
        - Do not drag concrete into position with vibrator.
        - Do not over vibrate. Remove vibrator when sound from machine changes.
        - Ensure good consolidation around the edges, in corners, around block outs and dowels. This is essential to prevent damage and yield proper shape. Allow mortar to flow into these areas under vibration without over vibration.
    - o Overfill slightly by 0.5 in. to give enough material to fill in surface while striking off. Screed as needed to ensure concrete is flush with top of form. Remove excess cut material from in front of screed as needed with the shovels.
        - Vibratory screed recommended over 2-in. x 4-in. lumber.
    - o Clean all materials used to place, consolidate and finish the slabs to this point.
    - o Verify the dowel bars are embedded at the correct depth into the slab using the preplaced marking on the dowel.
        - If short, pull dowel out to correct distance and reconsolidate with vibrator. If long, tap dowel into concrete with mallet.
2. Use a 4-ft bull float [with attached extension poles] and magnesium hand floats to level and fill the slab surface. (15 min per slab, 2 people)
    - o Start immediately after screeding to ensure all aggregate is pushed down and surface is level. Repeat as needed until material begins to set.
    - o Do not use finishing edgers to round over edges. Slabs are required to have 90 degree corners.
    - o Do not sprinkle or spray water onto concrete surface to assist with finishing. Light misting with a hand sprayer is acceptable.

- o A 3-ft steel fresno trowel [with attached extension poles] should be used if a good finish cannot be applied. This should be used if the surface voids are difficult to close. Let the concrete set about 5-10 min. after bull floating before troweling.
- o Clean the equipment used for this task and any remaining from previous tasks.

3. Install swift lifting anchor (Figure A11). (15 min total, 2 people)

- o Place steel when it cannot sink or float in the concrete. This will range from 30-60 min. after finishing. Stiffer mixes can be placed earlier.
- o Center the swift lift anchor over the indention made. Wiggle the anchor up and down lightly while inserting. Ensure the entire top of the plug is exposed (~1/8 in.) with the surface of concrete after all have been installed.

  - Float over the top of the swift lifting anchors lightly a magnesium hand float to fill in any holes around them. This will push the plug flush with the surface and cover the placed pieces.
  - When the concrete stiffens sufficiently, perform the final finishing techniques with the fresno float. This step pushes off any excessive water and removes surface deformations caused from the smaller hand finishing tools.

Figure A11. Swift lift anchor installation.

4. Texture slabs. (10 min total, 2 people)

- o Apply a non-skid texture to surface when the paste at the surface is strong enough to support this operation.
- o A broomed finish will be the easiest to apply.

5. Continue cleaning all equipment used.
6. Begin curing concrete slabs. (varies on method, 2 people)

    - The slabs can be cured by many different procedures once the bleed water disappears.

        - Flooding

            - Continuous saturating can be accomplished with a typical sprinkler and/or laying a sandbag dike around the perimeter of the slabs edged to contain the water.

        - Covering with plastic sheeting

            - Ensure entire surface is covered at all times.
            - Cut extra lengths to completely cover formed faces.
            - This method can discolor or stain the surface and has the potential to leave wrinkle marks when removed.
            - Apply burlap once surface texture can support weight without damage.

        - Double coating of curing compound with pump-up hand sprayer

            - No further work required if damaged application areas are repaired before film dries.
            - Spray twice where each spraying is perpendicular to one another.
            - CRD-300 (water-based) material required for Air Force and Army projects.

        - Double layer of wet burlap

            - Must maintain wet cure for entire cure period.
            - Apply burlap once surface texture can support weight without damage.

7. Finish cleaning all tools, equipment and area.

    - Long term storage of curing compound sprayer requires line flushing to prevent wax build up if applicable.
    - All tools should not be allowed to have heavy build up of concrete for proper construction of the panels.
    - Clean all spilt concrete on the pavement from around the formwork. This will make formwork breakdown much easier. A picture of a

panel cured in a form is presented in Figure A12 showing the locations of various form parts.

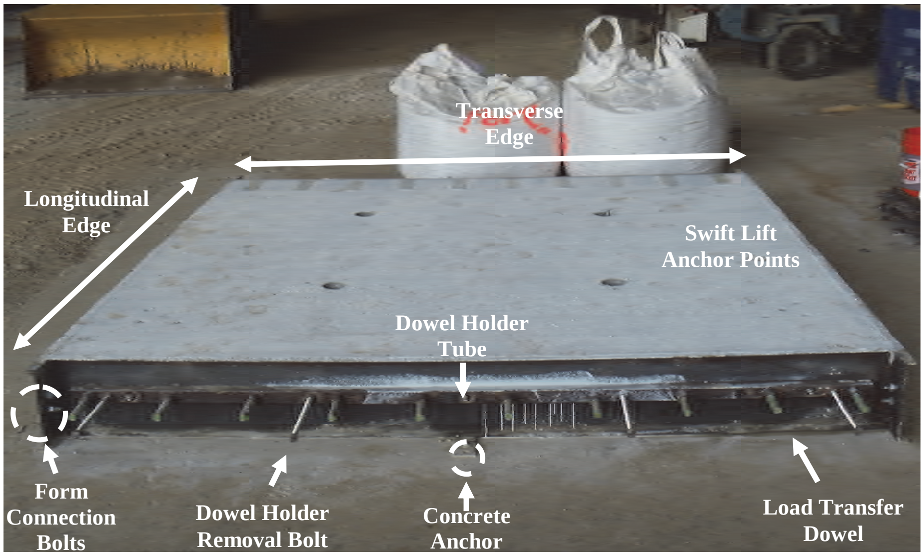

Figure A12. Cured panel in form.

Transport and storage

1. Remove formwork. (60 min total, 4 people)

- o Verify concrete curing time was achieved before starting work
    - 2,500 lb/in.^{2} compressive strength or 7 days of cure.
- o Remove concrete anchor bolts from pavement.
- o Remove form connection bolts used to connect form lengths.
- o Loosen the bolts that connect the formwork segments together.
- o Remove longitudinal form sections.
    - Use a mallet to strike the ends of each corner to break the bond between the PCC and the form.
- o Remove dowel bar accessories from the formwork.
    - **Use extreme care for this operation to prevent damage to the dowels and the PCC surrounding the dowels.**
    - Remove all bolts holding the steel blockouts to the forms (female side only).
    - Remove the bolts holding the dowel bar holders to the forms.
    - Install the removed bolts into the vertical holes of the dowel bar holder. Drive both bolts evenly with the impact hammer to assist

in pushing the dowel bar holder off the formwork. Use slow bursts (1-2 seconds) at a time over multiple passes. Continue driving until removed. Remove all dowel bar holders in a similar manner.

- Remove the separated formwork.
    - With the dowel bar holders removed, both pieces of separated formwork should slide over the dowels easily.
    - Spilt, unremoved concrete may be the only resistance to removal and will need to be removed.
- Remove steel blockouts. (male-female panel only) (30 min total, 2 people, 3 days later)
    - Insert a bolt into either bolt location on the exposed face of the steel blockout. Leave approximately 2 in. of the bolt exposed from the blockout.
    - Tap the bolt upward with a hammer to break the break the blockout free from the slab. Once debonded, the blockout should slide out easily (Figure A13).
- Trim the feathered paste from the top edges of the precast panel.
    - Use material that will not induce spall damage to the precast panel.

Figure A13. Removing the steel blockouts.

2. Remove the rubber recess form from swift lift. (10 min total, 2 people)
    - Obtain 2 screwdrivers from the tool kit.
    - Locate the two holes at the top surface of the plug.
    - Insert a screwdriver in each hole vertically.

- Rotate both screwdrivers simultaneously towards each other to pry the plug out.
- Remove any excess tape remaining in the recess.
- Clean any tape from the removed plug (Figure A14).

Figure A14. Rubber recess.

3. Apply curing method to slab in newly exposed areas. (15 min total, 2 people)

   - Apply curing compound to vertical faces, blockout holes and swift lift recesses.
   - Let concrete cure for a full 28 days before moving.

4. Transport slabs to storage area.

   - Obtain and position the crane.

     - Ensure crane has 15-ton minimum capacity.
     - Position the crane square with the panel to be lifted.

   - Check that all crew members are wearing all PPE equipment.
   - Draw paint lines on the doweled/blocked out vertical faces of the slab to mark the location of the storage dunnage. Paint lines should be drawn where the swift lifting points are, 30 ± 6 in. from the slab ends.
   - Attach the rigging to the crane.

     - Install a shackle at the end of 4 round slings.
     - Attach the round slings to the hook of the crane.

       - Install a lifting eye to each shackle. The large loop end receives the shackle. The round ball end attaches to the precast panel anchor as shown in Figure A15.

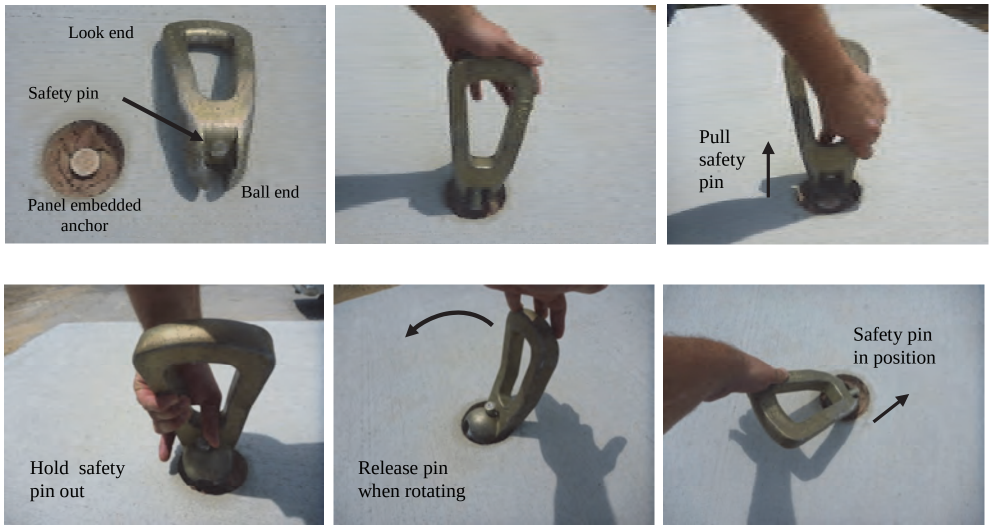

Figure A15. Installing lifting eyes.

- The lifting eye should bear on the shackle's bolt side.
- Ensure both the nut and safety clip are installed before lifting.

- Swing the crane line to the panel and safely center the line. Attach the lifting eyes to the embedded panel anchors.

  - Hold the lifting eye vertically, ball end down. Put the ball end of the lifting eye over the embedded anchor.
  - Find the sliding safety pin on the back of the ball and pull the pin vertically.
  - The "T"-shaped opening on the front of the lifting eye ball accepts the anchor. Slip the anchor into this opening. Continue rotating the lifting eye until the safety pin can fall into position and be flush with the ball.
  - Ensure the slings do not rub on any sharp corners during lifting to prevent wear and damage.

o Carefully lift the slab to an adequate height for the vehicle used for transport.

- Ensure the vehicle and/or trailer are rated to complete this work

        - Go no higher than 3 ft than the surface lifting over to allow ground personnel to rotate the aerial panel easily.

    - Place 2 pieces of dunnage on trailer at each edge and middle of slab and lower onto trailer.

        - Place dunnage 30 ± 6 in from slab edge, parallel to dowels.
        - Lower the panel safely onto the dunnage.
        - Secure load to trailer with appropriate load binding devices such as 5,000-lb ratchet straps. The ratchet straps should be placed so that they are directly over the lifting eyes of the panels.

5. Storing slabs at storage area.

    - Procure truck and trailer capable of hauling the panels based on size and weight.
    - Off loading at storage site is accomplished by the reverse of the on loading sequence.
    - Stack the slabs for storage.

        - Space dunnage to stack panels on ground.

            - All dunnage is placed 30 ± 6 in. from slab edge, parallel to dowels. This will be directly above and/or below the swift lift anchors.
            - The storage area should be level and solid ground. The area does not need to be paved, but granular surfaces should be sufficiently compacted or stabilized to ensure minimal uniform settlement over time.

        - Use two pieces of 4 in. x 4 in. x 10 ft pressure treated lumber.

            - In between each stacked slab.
            - Stack 4 slabs tall maximum.

Appendix B: Panel Installation Instructions

Phase 1: Prepare damaged pavement for precast slabs

This section documents the work required to prepare a damaged pavement for precast repair slabs.

1 Determine the areas where precast slabs should be installed.

- Currently, precast slab installation is used to replace corner portions of damaged in-place PCC pavements. Figure B1 documents the standard installation configurations.
- Slab centered or straddling joint(s) type installations may be more challenging or may severely damage the remaining slab to the point where full removal may be more advantageous.

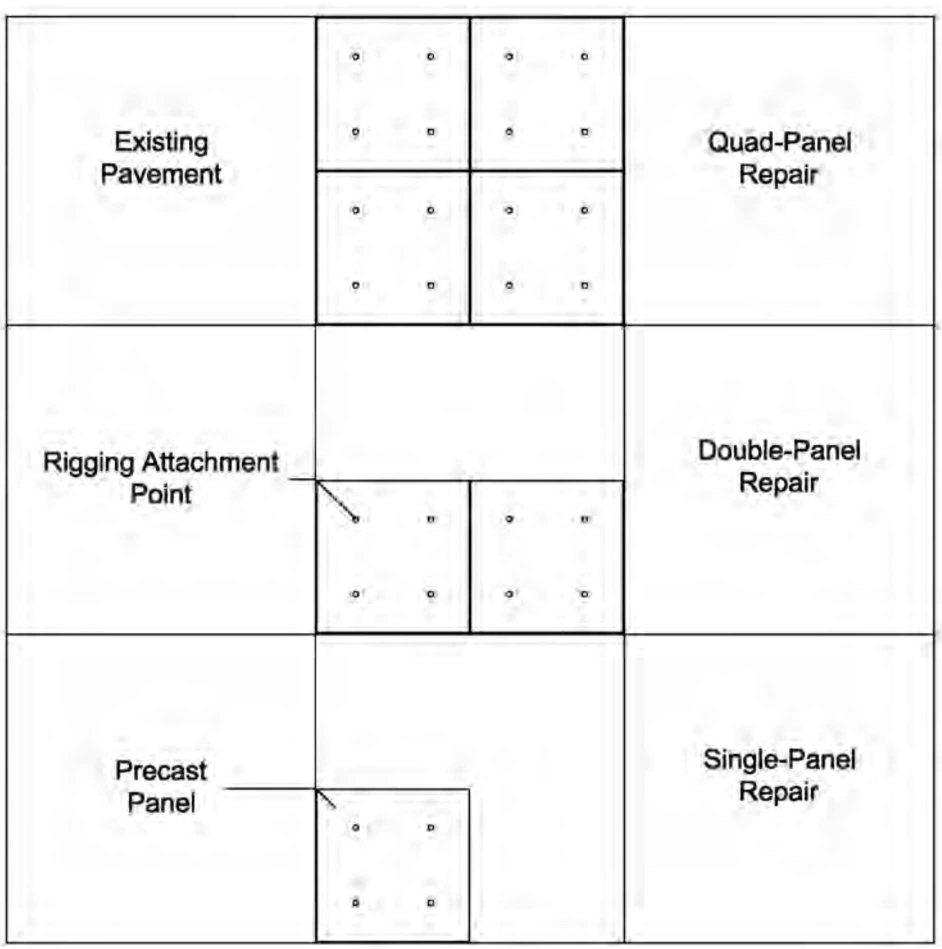

Figure B1. Precast panel installation configurations.

Procure equipment

% Make arrangements for a lifting crane.

- o Ensure a licensed operator familiar with the equipment is used.
- o Typical Air Force installations have access to a 15-ton mobile crane.
  - A 15-ton crane is adequate to conduct a single-panel replacement when setup directly in front of the removed area.
  - Multiple setups will be required to conduct double- and quad-panel repairs due to the allowable crane lifting radius. Installation times increase significantly from crane repositioning.
- o A minimum 30-ton crane is recommended to complete double and quad-panel replacements utilizing a single crane setup.
- o Any type of forklift equipment is not currently recommended due to operational and personnel safety and to prevent pavement damage.

2. Make arrangements for a concrete floor saw and concrete saw blades.

- o Walk-behind saw must be capable of performing full-depth saw-cutting operations.
- o Determine appropriate saw blade diameters to exceed the in situ pavement thickness.
  - Multiple saw-blades, progressively increasing in diameter, are recommended to perform saw cutting operations.
  - Ensure at least one back-up blade for each blade size.

Mark distressed area

%CMark areas requiring repair work.

- o Mark damaged areas slated for removal
  - Use a chalk line tool and to delineate the area(s) to be removed.
    - Single-panel replacement
      - ▪ A 10-ft ± 1/8 in. square area must be removed for this slab. This allows for a typical 0.75-in.-wide construction joint surrounding the precast panel for ease of installation.

        - Failure to accurately square and mark removal area can significantly complicate installation of precast repair panels.

    - Double-panel replacement

        - A 20-ft ± 1/8 in. x 10-ft ± 1/8 in. area must be removed for this slab.
        - The removal section should be marked so that the 20-ft x 10-ft section is divided into two 10-ft x 10-ft sections.
        - Failure to accurately square and mark removal area can significantly complicate installation of precast repair panels.

    - Quad-panel replacement

        - A 20-ft ± 1/8 in. x 20-ft ± 1/8 in. area must be removed for this slab.
        - The removal section should be marked so that the 20-ft x 20-ft section is divided into four 10-ft x 10-ft sections.
        - Failure to accurately square and mark removal area can significantly complicate installation of precast repair panels.

  - Paint a line over the chalk line to mark the saw cut locations.

    - Use waterproof paint to ensure paint line remains visible during saw-cutting operations.
    - Use the paint line template to make a neat 1/8-in. line.

  - Mark a number next to the repair. This number will be used for labeling the location of the replacement slab.

- Mark dowel receptacles in adjacent parent slabs

  - Dowel receptacles will be constructed in the parent slab (section not slated for removal).
  - Align dowel receptacle template along paint line (transverse edge(s)) of previously marked slab.

    - Use the dowel receptacle stencil and waterproof paint to mark the locations for the dowel receptacles. Fully paint the areas in between the teeth to indicate what will be removed.

- All dowels will be parallel to traffic, i.e. longitudinal direction.
- Ensure the edge of the template is flush with the transverse edge of the paint line of the marked slab.
- Marking the correct location is critical for the future installation of the repair. Ensure all measurements are correct.

Saw cut distressed area

1. Use a concrete saw to cut the marked locations (Figure B2)

   o If time permits, use a series of passes incorporating progressively larger blades to ensure a straight, clean cut. Suggested cut depths and blade diameters are provided in Table B1. In addition, the maximum cut depths for a variety of blade diameters are provided in Table B2.

     - A skilled saw operator is recommended.
     - Each pass should increase the cut depth by 1/3 to 1/4 of the total slab depth.

Figure B2. Saw-cutting distressed slab and dowel receptacles.

Table B1. Suggested cut depths and blade diameters.

PCC Depth	Pass Number	Cut Depth	Recommended Blade Diameter
12-17 in.	1	3-4.5 in.	24 or 26 in.
	2	6-9 in.	26 or 30 in.
	3	9-13.5 in.	36 in.
	4	12-17.0 in.	36 or 42 in.

Table B2. Suggested cut depths and blade diameters.

Blade Diameter, in.	Maximum Cut Depth, in.
24	8.5
26	10.5
30	12.5
36	15.0
42	17.5

- Limit corner cross-cuts to a distance approximately equal to the slab depth.
- Straight cuts are critical to installation speed and the fit of the repair. Cutting to deep to fast will make wavy (horizontally) cuts that make removal and the future repair installation difficult.
- Ensure full-depth penetration.
    - Upon full penetration, the water discharged from the saw typically assumes a brown, muddy look.
    - The saw blade resistance also typically decreases noticeable when the blade enters base course material.

o Larger slab removal

- Start with a minimum blade diameter of 24 in. for the first pass. For additional passes, progress to 30 in., 36 in., and a final blade diameter if required to achieve a full depth cut.
- The final blade diameter used depends on the thickness of the damaged pavement.
- Verify the full-depth penetration with a thin piece of wire at each corner (Figure B3).

o Dowel blockout

Figure B3. Verifying full-depth saw cut.

- Place center of saw blade at the back of the marked dowel receptacle (12 in. from perimeter of saw cut slab).
- Insert saw blade to a total depth of 6.75 ± 0.25 in. Do not attempt to cut the width at the back surface of the dowel.
- Terminate saw cutting operations when saw blade is centered at the joint with the damaged pavement.
- Verify the cut depth with a piece of wire.

Slab removal

1. Remove the damaged slabs.

   - o If the damaged slab is shattered into multiple pieces, the entire slab must be broken by demolition equipment.

     - Use a skid steer or excavator with an impact hammer attachment to break the concrete slab into manageable pieces.
     - Remove the broken concrete with an excavator and place into a dump truck for removal.

   - o If the damaged slabs are not broken into multiple pieces, concrete expansion anchors can be used to lift the slab from its location

     - Install the expansion anchors.

       - Mark the location of the expansion anchor holes using paint

    - Holes can be located from 24 to 36 in. square from the corner depending on the damage to the slab. If possible, the anchors should be installed at located 30 in. square from the corner to balance load capacity and slab stability when lifting.
    - All anchors must be installed at the same distance from each corner.

- Use a rotary hammer and 1.25 in. concrete drill bit to drill the expansion anchor holes (Figure B4).

    - Align the hole drilling jig over the drilling location. The operator should stand on the base of the jig to ensure it does not move.
    - Ensure the drill bit makes a fairly vertical cut (± 5° of vertical) for easy installation. Use caution when removing and reinserting the bit to ensure a vertical cut is made. Cutting different angled or multiple paths will make anchor installation very difficult. Use of the hole drilling jig will achieve the vertical cut automatically.
    - Do not attempt to widen the hole to make anchor installation easier. This will cause the anchor to fail when activating and not hold the concrete.
    - Drill to a 12-in. maximum depth. The jig is designed to allow for a cut this deep if a 17-in. long bit is used.
    - Clean the hole with blasts of compressed air.

Figure B4. Drilling expansion anchor hole.

- Prepare the expansion anchor (Figure B5).
    - Remove the nut and washer from the anchor.
    - Place the anchor's threaded rod through the swivel hoist ring.
    - Place the bushing that fills the center.
    - Ensure the cone base is flush with the threaded rod.
    - Replace the washer and nut. Twist the nut finger tight. Go no more than 1/4 turn past finger tight. Further tightening will activate the anchor and either damage it or prevent its installation.

Figure B5. Expansion anchor preparation.

- Install the anchor (Figure B6).
    - Place the anchor into the hole, anchor side first.
    - Place the setting tool through the threaded rod and drive the anchor into the concrete with a hammer until the swivel hoist ring is flush with the concrete surface.
        - If the hole is not drilled within the specified vertical angle tolerance, driving the anchor into the concrete may be extremely difficult.
        - The anchor will be difficult to remove even installed properly before activation.


Figure B6. Installed expansion anchor and swivel hoist ring prior to tightening.

- Activate the expansion anchor in the concrete
    - Use an impact wrench to tighten the nut of the anchor. Only tighten for 5 seconds to ensure the required installation torque is not exceeded (Figure B7a).
    - Use a torque wrench to continue tightening the nut until 240 ft-lbs is reached is reached (Figure B7b).
    - Trim the threaded rod with the portable band saw if full movement of the swivel hoist ring is hindered as needed. The current specified anchors should need minimal to no trimming after activation (Figure B7c).

o Remove the slabs (Figure B8).

- The crane's capacity depends on its location. The larger capacity crane available from the minimum of 15 tons, the easier it is to place double- and quad-repair panels from one location. It is highly recommended to use a larger capacity crane if available. If the minimum capacity crane is used. Set the crane up square with the cutout section along one of its edges.
- Position the crane directly in front of the removal area.
- Install a shackle at the end of 4 round slings. This will help prevent tangling of the slings when attaching to the crane.
- Attach the round slings to the hook of the crane.
- Install each shackle to the swivel hoist ring.

Figure B7. Activating the expansion anchor.

Figure B8. Lifting slab.

        - The swivel hoist ring should bear on the shackle's bolt side.
        - Ensure both the nut and safety clip are installed before lifting.

    - Lift the slab slowly until completely removed.
    - Place removed slab on truck for removal.

- Prepare the receptacles in the adjacent and parent slabs

    - Use a jackhammer with the 3-in. chisel and remove the receptacles (Figure B9).

        - Start at the back of the receptacle and place the chisel tip at the painted boundary. Try to break the concrete out in a single large piece.

            - Let the hammer do most of the work to conserve energy. Start vertical until slightly into the concrete, and then begin angling back 30° from vertical.

        - Ensure the receptacle is rectangular in shape with even dimensions across each side. Additional bits were provided to assist with cleaning and leveling as needed.

            - A prismatic shape is critical for the performance of the repair material that will fill this area (Figure B10).

    - Blow the receptacle out with compressed air.
    - Use a steel brush and scrape the chiseled surfaces to remove any loose material and dried saw slurry. Blow the receptacle out again with the air lance.
    - Use a sponge and water to remove any sawing slurry from the blockout. Blow the receptacle out again with the air lance.

        - For a more permanent repair, water or sand blasting equipment could be used to prepare the receptacle surfaces.

Figure B9. Receptacle excavation with 3-in. chisel bit.

Figure B10. Using bushing bit to produce prismatic blockout.

Phase 2: Installation of precast slabs

Installation preparation

1. Prepare sub base, if required.
    - If using the lift out method of damaged pavement, this will limit disturbance to the sublayers.
    - If sub base requires repair, remove disturbed or weakened material and replace with suitable backfill.
2. Make preparations for base material.

- Flowable fill will require equipment and supplies stationed at the installation or off-base arrangements.
- Rapid-setting flowable fill mixtures are highly recommended to ensure speedy reopening times.
    - The mixture design compressive strength required is based on projected aircraft use. Recommended minimum values at time of opening are:
        - C-17: 85 $lb/in.^2$
        - F-15E: 55 $lb/in.^2$
    - 20 min of working time is recommended for single-panel repairs.
    - 40 min of working time is recommended for double-panel repairs.
    - Care should be taken when selecting set times for quad-panel repairs. Set times should allow for adequate time to install and seat all panels. A total of 60 min of working time is recommended if panels are installed as two double-panel repairs.

3. Install bridge plates on precast slabs.
    - Bridge plates are designed to attach and cantilever from the precast panel when installed. After installation, the cantilevered section will bear on the adjacent existing pavement to ensure precast panels are installed flush.
    - Select a slab to be installed at a particular location. Mark its intended location (number previously painted on pavement) with paint.
    - Collect the correct number and type of bridge plates for the planned slab installation (Figure B11).
        - Single and double: 4 short plates
        - Quad: 2 short and 1 diagonal plate, 1 concrete disk
    - Install the concrete anchors
        - Align the anchor template with the corner of the slab to have the holes face the correct direction. Use a lumber crayon to mark the drilling locations of the small concrete expansion anchors on the precast panels.

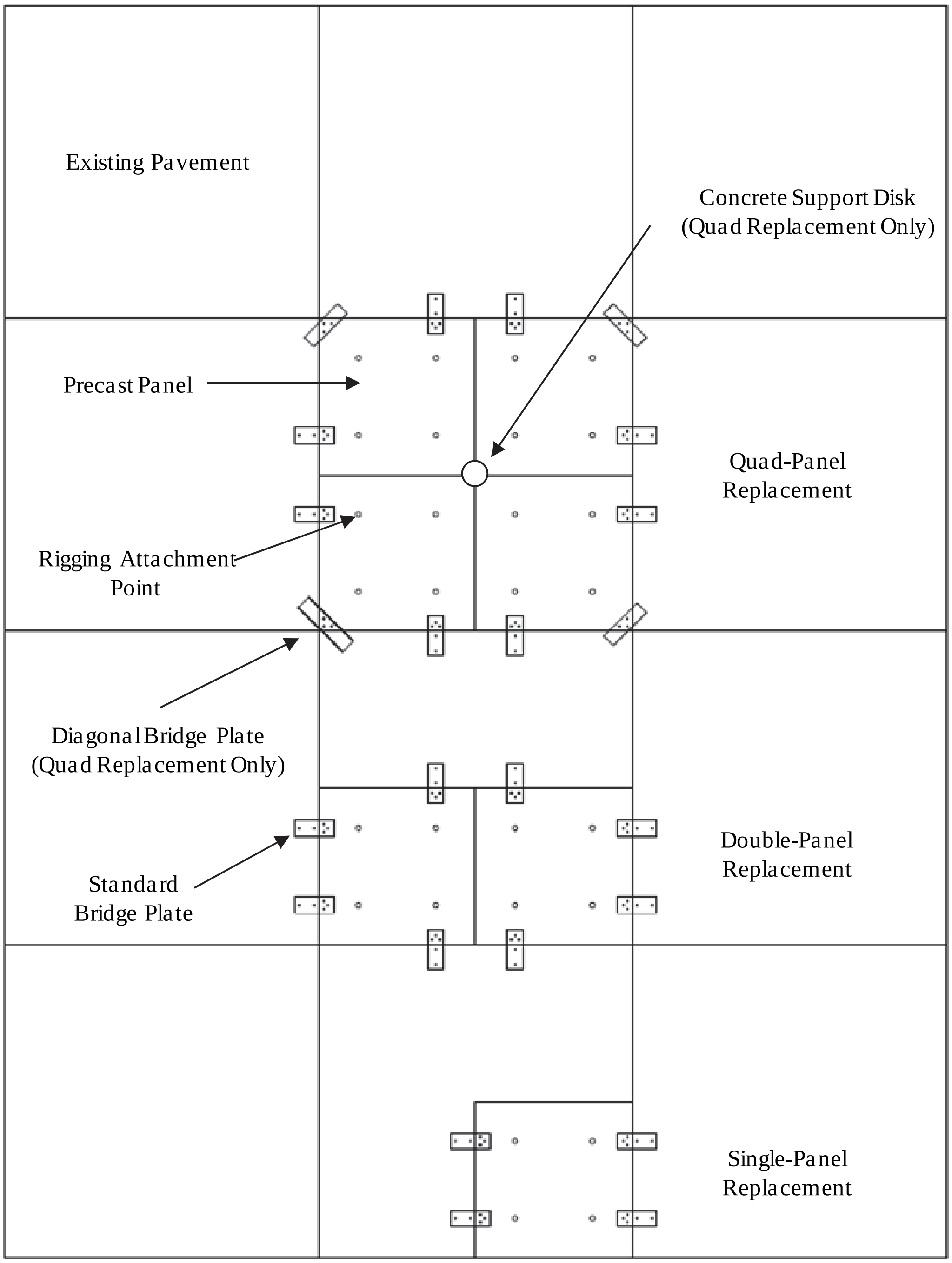

Figure B11. Bridge plate configurations for various repair types.

- It is essential that the template is used to ensure the correct location of the anchors. Placing the anchors in the incorrect location will cause the anchors to fail (pull out of concrete) and not support the slab.

- Drill holes into the concrete at the marked locations using the hammer drill with a 0.5-in. masonry drill bit (Figure B12). Drill to a depth of 6 ± 1/8 in. Clean the holes with compressed air.
- Install the small concrete expansion anchors into each hole. The cone end goes first and the nut and washer should not be in place. Drive the anchor into the concrete with a hammer.

    - Do not preinstall the nut to the threaded rod of the anchor. Hammering on the nut when installed damages the threads of the threaded rod and prevents tightening of the nut during installation.

Figure B12. Bridge plate concrete anchor installation.

- o Secure the bridge plate.

    - Bridge plate installations should be completed simultaneously with base layer preparation to maximize repair efficiency tempo.
    - Place the bridge plate over the installed concrete anchors (Figure B13). Ensure the correct alignment is made for the repair type and direction.
    - Place a square washer over each installed concrete anchor.

Figure B13. Attaching bridge plate to precast panel.

- Install the expansion anchor washer and nut. Tighten the nut until hand tight. Use an impact hammer to continue tightening the nut for 3 seconds (Figure B13)

o Use a torque wrench to continue tightening the nut to 55 ft-lbs of torque.
o Photos of the installed bridge plates are provided in Figure B14.

Figure B14. Bridge plate layouts.

4. Prepare the base layer.

o Ensure delivery of flowable fill.
o Prepare the aluminum screed.

    - Set the height of the screed to allow for an additional 1/16 in. of material more than what is needed to ensure the void between the replacement slab and underlying granular material is filled.

  - Use a tape measure and mark the 11-in. depth of the repair slab around the entire interior perimeter of the hole. This indicates the height of flowable fill needed.
  - Place the support disk (Quad-panel repair only)

    - Draw string lines to locate the position of the interior corners.
    - Dig a 2-ft square x 1-in. deep hole centered at the string line intersection. Replace and overfill with a half a bag of sand. Hand level the sand surface (Figure B15).
    - Place a precast concrete disk on the sand in each string line intersection quadrant. Twist the disk back and forth to lower into the sand. Use a tape measure to install the surface of the disk 11 in. below the string line.

Figure B15. Placing a concrete support disk.

Precast panel installation

1. Basic/single-panel repair installation.

- Position crane at repair site.

  - Crane should be pre-positioned prior to or as base layer material is placed.
  - Install a shackle at the end of 4 round slings.

- Attach the round slings to the hook of the crane.
- Install a lifting eye to each shackle. The large loop end receives the shackle. The round ball end attaches to the precast panel anchor.

  - The lifting eye should bear on the shackle's bolt side.
  - Ensure both the nut and safety clip are installed before lifting.

- Swing the crane line to the panel and safely center the line. Attach the lifting eyes to the embedded panel anchors (Figure B16).

  - Hold the lifting eye vertically, ball end down. Put the ball end of the lifting eye over the embedded anchor.
  - Find the sliding safety pin on the back of the ball and pull the pin vertically
  - The "T"-shaped opening on the front of the lifting eye ball accepts the anchor. Slip the anchor into this opening. Continue rotating the lifting eye until the safety pin can fall into position and be flush with the ball.

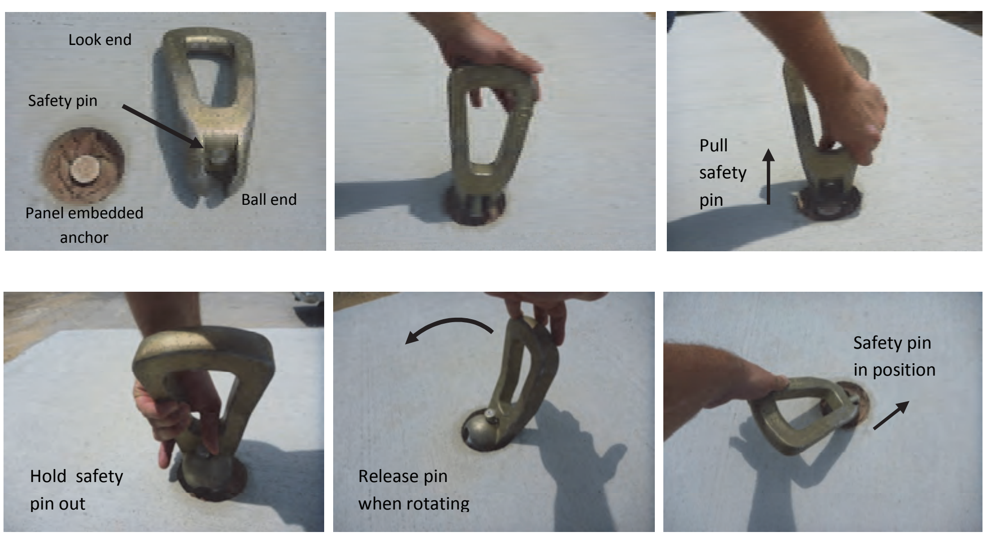

Figure B16. Attaching lifting eyes to embedded panel anchors.

- o Place the base layer material.
  - Place enough material to fill entire 10-ft x 10-ft excavated area to 11-in. depth mark. Be careful not to overfill as excess must be removed.
  - For portland cement based materials, use a concrete rake and shovels to spread the material to reduce segregation.
- o Screed the placed material.
  - Remove excess material in front of screed as it is pulled (Figure B17). Keep the amount of material in front of screed at about 1-2 in. to ensure no low spots.
  - Place additional material in low areas and rescreed.
  - Screed the repair at least twice to ensure the correct elevation is achieved. Make screedings perpendicular to one another to ensure smoothness to ensure the surface follows grade, if possible.
  - Make sure to remove as much excess material as possible to ensure panel can be fully seated flush with surrounding pavement and to limit material from being forced into construction joints and dowel receptacles.

Figure B17. Screeding base layer material and removing excess flowable fill.

- o Install the replacement slab.
  - Lift the slab (Figure B18) and center over the repair. Maintain a lifting height of less than 3 ft to ensure the slab can clear any extended crane outriggers and so the installation crew can easily manipulate the aerial panel.
  - Rotate the slab as needed to have the bridge plates in the correct positions.

Figure B18. Precast panel prepared for installation.

- Position personnel in each corner of the slab with the correct thickness of shim spacers. Use 2 sets of spacers at each corner. Typical spacer thicknesses are 3/4 and 3/8 in. for joints without and with expansion board installed, respectively.
    - Three different sizes of shims are recommended as needed to achieve the necessary joint widths.
        - 1/8 in.
        - 1/4 in.
        - 1/2 in.
- Slowly lower the slab into place. Shims should be held against the corners of the replacement slab to ensure the proper joint width between the precast panel and the surrounding parent slab.
- Use a 6-ft level to verify the flushness (checking for faulting) of the surface elevation across the joints.
    - If the panel will not lower into the repair evenly (greater than 1/8 in.), use a vibratory roller (Figure B19) to assist with seating the panel. Place plywood beneath the roller to protect the precast panel and surrounding pavement. Pulses of vibration are significantly more helpful than only applying weight.

Figure B19. Using vibratory roller to fully seat precast panel.

Figure B20. Removing swift lifting eyes.

    - Shims can be placed directly beneath the bridge plates to achieve flushness as needed as a last resort. Panel may need to be lifted slightly to install shims beneath bridge plates.

  - Remove the swift lifting eyes.

    - Push the safety pin in.
    - Rotate the lifting eye until free.

  - Remove any excess flowable fill from the receptacles. This is significantly easier before it sets.

- o Fill in the receptacles in parent slab (Figure B21).

  - Push the expansion joint board against the slabs with receptacles.

Figure B21. Filling receptacles.

- Use shim spacers to evenly push and hold the board against the face of the joint.
- Remove any excess base layer material from the joints as needed.

- Remove any base layer material from receptacles
- Blow the receptacles out with compressed air.
- Wipe off the dowel bar to ensure cleanliness and good bond with a wet sponge.
- Begin making receptacle filling material.
    - Proprietary rapid-setting products are expected to be utilized for this task.
        - Guidance on selecting acceptable materials can be found at https://transportation.wes.army.mil/triservice/pavement_repair.aspx. Products used should have speedy strength gains and will fall under either the “Crater” or “Primary Airfield Repair” categories listed. Any aggregate extended aggregate should use #67 sized (1.0 in max or less).
- Fill and level all the receptacles and formed anchor depressions.

- Ensure the expansion joint board effectively dams up the receptacles. The gorilla glue will help seal the blockout.
- Clean any heavy deposits of foamed gorilla glue with a razor blade before filling.
- Overfill by 1/16 in. for potential shrinkage. Minimize splatter and excess runoff around the blockout. Use a trowel to finish the material.

o Bridge plate removal (Figure B22).

- The bridge plates can be removed once the base layer material gains at least 5 lb/in.^{2} of compressive strength.
- Remove all the nuts and washers with the impact wrench. Some nuts may need to be cut off with the bandsaw if difficult to remove.
- Use a grinder with a metal cutoff wheel to cut the exposed anchor thread off flush with the surface. Grind into the anchor hole as necessary to remove burrs.
- Use a steel punch and hammer to drive the anchor into the drilled hole to ensure the anchor remaining is not a tire hazard.

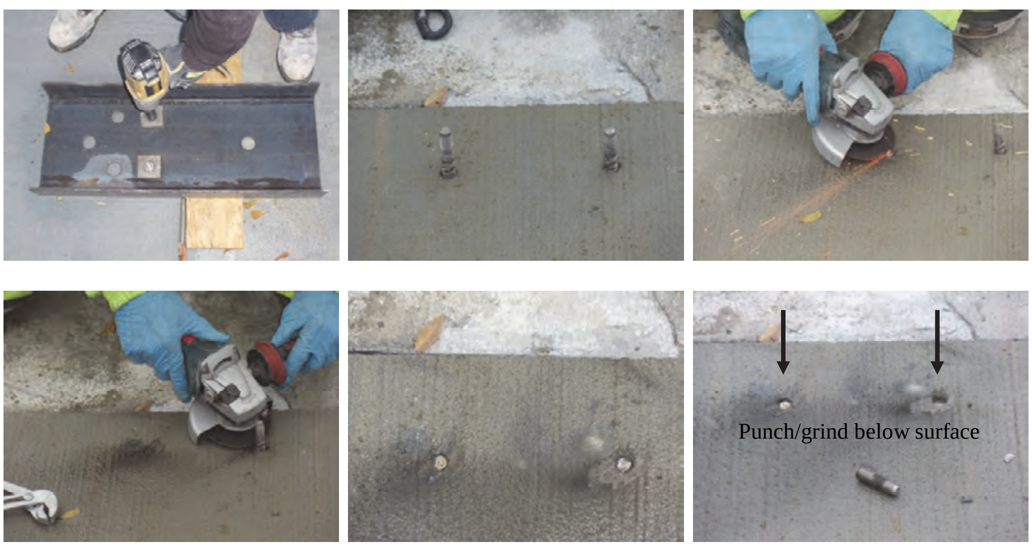

Figure B22. Bridge plate removal.

o Install joint sealant.

- Install an oversized backer rod in the joint opening.
  - For silicone sealant, the top of the backer rod should be at a depth of half the joint width plus 1/8 in.

- Use a pneumatic joint sealant gun to apply the joint sealant.
    - Maintain the surface of the sealant 1/8 in. below the pavement surface.
- o Allow flowable fill and dowel receptacle repair material materials to gain strength.
    - A dynamic cone penetrometer (DCP) can be used to monitor the approximate bearing capacity of the flowable fill placed. The design strength must be achieved before reopening.
    - Dowel receptacle repair material strength should reach a minimum unconfined compressive strength of 3,000 lb/in.^{2} prior to opening to traffic.

2. Multiple-panel installation. Installing multiple panels within a damaged existing slab involves the majority of the tasks used with the single-panel installation described earlier. Any differences in the installation procedure and tips for success are given below.

- o Double-panel installation.
    - Position crane at repair site.
        - If the minimum sized 15-ton crane is used, installation must be accomplished with the crane squarely positioned in front of the installation site due to the crane's allowable lifting capacity at the radii used. The crane must be moved after each installation for safe and efficient use.
        - Cranes with capacities greater than 30 tons can be positioned squarely anywhere around the installation site for one setup. Once setup allows for little lost time due to repositioning the crane.
    - Place and screed the base layer material.
        - Place enough material to fill entire 20-ft x 10-ft excavated area to the 11-in. depth mark.
        - Screed additional panel areas perpendicular to the previously placed panels.
    - Install the first precast panel.

- Ensure a tight fit at the corners. An excessive construction gap between panel 1 and the adjacent parent slab decreases the construction gaps between the proceeding panels; which can result in damaging the precast panels and/or the parent slab during the installation process.
- Verify the panel is fully seated on the base layer material and there are no apparent voids beneath the precast panel. No voids will be present along free edges if sufficient material is placed before installation. Excess flowable fill will move to the areas where additional slabs will be placed.
- Re-screed the base layer material over the intended installation site before installing a panel.

- Install the second precast panel.
  - Care must be taken when placing additional panels around installed panels.

- o Quad-panel installation.
  - Position crane at repair site.
    - The first panel delivered should be a terminal panel to accommodate connecting a standard panel. The connecting standard panel should be completed after a terminal panel to facilitate screeding. Continue the pattern for installing the second half of the repair. Both terminal panels will be diagonal of each other when installed.
  - Place the base layer material.
    - Place material in corner of excavated section where panel 1 is to be installed (Figure B23).
      - Approximately 50 percent of total calculated base layer material should be placed in this corner.
      - The screed cannot be used to remove excess material when installing the first panel since the span distance is too long. Use a concrete rake and shovels to spread the material to sufficient height in this region to fully seat the first panel.

Figure B23. Placing initial base layer material and first panel for quad-panel installation.

- Install precast panel 1.
    - Ensure a tight fit at the corners. An excessive construction gap between any of the slabs decreases future construction joints between slabs; which can result in damaging the precast panels and/or the parent slab during the installation process.
    - The interior corner of the precast panel should be bearing on a support stone.
    - Verify the panel is fully seated on the base layer material and there are no apparent voids beneath the precast panel.
- Install precast panel 2.
    - Panel 2 should be a standard panel oriented such that the load transfer dowels fit into the previously installed precast panel 1 receptacles on one side and the parent slab receptacles at the opposite end
    - Screed the material in the region where the second panel is to be installed.

    - Install the second precast panel.

- Install the remaining panels.

    - The process is similar to steps for panels 1 and 2.
    - Add additional base layer material to the third precast panel repair area.

        - Approximately 90 percent of the batched flowable fill should be placed in excavated region when ready to install the third panel.
        - The remaining material will be placed after the third panel has been installed and limits removing excess.

    - Screed the base material and install the male-female panel.

Appendix C: Precast Panel Kit Equipment/ Material List

Small Equipment Listing

Number	Quantity	Item	Manufacturer	Model Number
Container, shelving and organization				
1	1	20 ft Cargo container		
2	3	industrial shelf units (1 ton capacity)		
3	5	rolls of Velcro straps		
4	2	fire extinguisher - multipurpose		
5	1	first aid kit, 140+ pieces		
6	7	1800 in^3 canvas tool bag, zipper, water resistant		
Hand tools				
1	2	automatic rebar tying tool		
2	2	rebar tying pliers		
3	2	16 oz rubber mallet		
4	2	torque wrench, 0.5 in chuck, 250 ft-lb capacity		
5	1	big snap chalk line tool		
6	1	6 ft aluminum level		
7	1	6 ft level case, hard plastic		
8	2	razor blade knives, extra blades included		
9	1	rough surface floor broom		
10	3	1 gal hand sprayer		
11	2	bucket opener		
12	2	wire scrub brush (head)		
13	4	wire brush- long handled		
14	2	25 ft measuring tape		
15	1	300 ft fiberglass measuring tape		
16	2	3 lb drilling hammers		
17	4	2.5 in. wide carbide scraper		
18	4	2 in. wide stiff putty knife		
19	12	5 gal plastic bucket		
20	1	48 in. magnesium channel bull float, with ez tilt knuckle	Kraft	
21	1	36 in. steel fresno float, with ez tilt knuckle	Kraft	

Number	Quantity	Item	Manufacturer	Model Number
22	6	6 ft aluminum extensions poles for bull and fresno float	Kraft	
23	2	24 in. concrete broom head		
24	2	wooden concrete broom pole		
25	2	18 in. magnesium hand floats		
26	1	30 in. pry bar		
27	1	48 in. pry bar		
28	1	manual rebar cutter	Hit Tool	22-RC19
29	4	heavy duty 3/4 in. garden hose - 100 ft	Gilmour	Flexogen
30	1	heavy duty garden hose reel with swivel - 400 ft capacity	Jackson	
31	1	heavy duty garden hose reel - 150 ft capacity		
32	1	water hose nozzle		
33	1	brass Y garden hose adaptor		
34	2	long handled scrub brush		
35	1	10 lb sledge hammer		
36	1	large pick		
37	2	hand brooms		
38	1	concrete rake		
39	2	digging shovel		
40	2	square shovel		
41	2	sand rake		
42	2	5 gal gas can		
43	1	inverted marking paint spray gun		
44	2	spray paint trigger nozzles		
45	1	Essential tool kit		
		Power tools		
1	1	6500 W generator	Honda	EM6500SX
2	1	3000 W generator	Honda	EU2000IS
3	1	wheel kit for 3000W generator	Honda	-
4	4	25 ft extension cords, 12 gauge, 15A		
5	4	50 ft extension cords, 12 gauge, 15A		
6	8	extension cord reel - 150 ft capacity		
7	1	2 HP concrete vibrator motor	Multiquip	CV-2
8	1	7 foot flexible concrete vibrator shaft	Multiquip	382V-FS7
9	1	3 foot flexible concrete vibrator shaft	Multiquip	382V-FS3
10	1	1.625 in. steel concrete vibrator head	Multiquip	1700HD

Number	Quantity	Item	Manufacturer	Model Number
11	1	1.375 in. steel concrete vibrator head	Multiquip	1400HD
12	2	1.7 hp hand mixer, single shaft with paddle quick connect	Collomix	CX600HF
13	2	mortar mixing paddle	Collomix	MK160HF
14	2	concrete hand mixer paddle	Collomix	MK140HF
15	1	Portable band saw	Dewalt	DWM120K
16	3	0.5 in. chuck hammer drill	Dewalt	DWD520K
17	2	4.5 in. angle grinder	Dewalt	D28402K
18	2	0.5 in. drive impact hammer	Dewalt	DW292K
19	1	4.5 gal air compressor	Dewalt	D55146
20	1	rotary hammer drill - SDS-MAX chuck	Dewalt	D25602K
21	2	infrared thermometer		
22	2	joint sealant gun	Cox	61002
23	2	air hose quick connect plug, female connection		
24	2	air hose quick connect coupler, female connection		
25	2	3/8 in. diameter , 50 ft long rubber air hoses		
26	2	extension wand for 3/8 in. air hose		
27	1	6.5 hp shop vacuum		
28	2	replacement heavy duty vacuum tube for shop vacuum		
Rigging equipment				
1	4	7/8 in. shackles	Crosby Group	1019837
2	10	safety cotter pin 0.91 in x 2.75 in		
3	8	swivel hoist ring	Crosby group	1016975
4	8	bearing for swivel lifting hoist	Custom - see drawing	
5	4	4 ton swift lifting eyes	Dayton Superior	60576
6	16	Swift lift plus recess plug	Dayton Superior	121046
7	6	8 ft synthetic endless round sling		
8	8	2 in. wide ratchet tie downs, 3 kip min capacity, 10 ft long		
Custom formwork and installation equipment				
1	4	Formwork - Side A		
2	4	Formwork - Side B		
3	2	Formwork - Side C		
4	48	side bridge plate		

Number	Quantity	Item	Manufacturer	Model Number
5	12	corner bridge plate		
6	1	collapsible aluminum concrete screed		
7	1	collapsible flowable fill screed		
8	2	large concrete anchor drilling jigs		
9	60	1/8 in. shims		
10	60	1/4 in. shims		
11	60	1/2 in. shims		
12	18	bolt-on form blockouts		
13		0.5 in. hex bolts		
14		0.5 in. hex nuts		
15	250	bridge plate washer		
16	1	Lexan dowel receptacle stencil		
		Essential tool kit		
1	1	1800 in^3 canvas tool bag, zipper, water resistant		
2	1	rafter/speed square		
3	1	framing hammer		
4	1	3/8-in.drive socket set - 30 piece		
5	1	regular pliers		
6	1	needle nose pliers		
7	1	cutoff pliers		
8	1	linesman pliers		
9	1	arc pliers		
10	1	12-in.crescent wrench		
11	1	8-in.crescent wrench		
12	1	screwdriver set - 8 piece		
13	1	10-in.locking pliers		
14	1	7-in.locking pliers		
15	1	5-in.locking pliers		
16	1	set of hex wrenches		
		Safety gear		
1	1	neoprene chemical gloves - M		
2	2	neoprene chemical gloves - L		
3	2	neoprene chemical gloves - XL		
4	4	sets of knee pads		
5	8	safety glasses		
6	2	face shields		

Disposable Supplies Listing

Number	Quantity	Item	Manufacturer	Model Number
Slab construction				
1	220	#3 plain rebar, grade 60 - cut to 9 ft 7.5 in. long		
2	75	#3 plain rebar, grade 60 - cut to 1 ft long		
3	140	#5 plain rebar, grade 60 - cut to 9 ft 7.5 in. long		
4	220	1 in. x 20 in. A615 grade 60 steel dowels, epoxy coated with ends patched, no bond breaker		
5	52	4 ton swift lift precast anchors , 0.75 in. x 7.125 in.	Dayton Superior	60635
6	300	1.5 in. bar chair	Dayton superior	73260
7	200	6 in. high chair	Dayton superior	75430
8	250	8 in. 16 gauge rebar ties		
9	1000	6 in. 16 gauge rebar ties		
10	10	permanent marker - fine tip	sharpie	fine
11	1	5 gal bucket of multipurpose grease		
12	2	5 gal box of form release		
13	4	roll of duct tape - 35 yd		
14	4	10 ft wide x 100 ft long roll of plastic sheeting		
15	1	box of disposable gloves		
Installation				
1	24	4 in. square by 10 ft long pressure treated lumber		
2	3	5 gal bucket of curing compound (with screw top cap)		
3	3	4 ton swift lifting eye replacement safety pins	Dayton Superior	
4	25	10 ft long, 8 in. wide, 0.375 in. thick fiber expansion joint board		
5	1	0.75 in. closed cell foam backer rod		
6	1	1 in. closed cell foam backer rod		
7	2	1.25 in. closed cell foam backer rod		
8	72	self leveling silicon sealant - 29 oz_f cartridges	Tremco	Spectrum 900SL
9	6	marking crayons		
10	56	Concrete sleeve anchor	Simpson	TCAP751458
11	3	Concrete sleeve anchor setting tool	Simpson	TCAP75
12	300	concrete expansion anchor	Redhead	WS1270G
13	10	4.5 in. metal grinding wheels	Dewalt	DW4523
14	10	4.5 in. masonry/concrete grinding wheel	Dewalt	DW4429
15	3	portable band saw replacement blades	Dewalt	DW3986
16	2	1.25 in. diameter concrete drill bit (SDS-MAX chuck), 17 in. long		

Number	Quantity	Item	Manufacturer	Model Number
17	10	0.5 in. diameter concrete drill bit (regular chuck), 12 in. long	Dewalt	DW5236
18	1	bottle of powdered chalk		
19	1	roll of masonry string		
20	8	sponges		
21	30	cans of orange marking paint		
22	16	cans of yellow permanent spray paint		
23	4	quart paint measuring cup		
24	8	36 lb sealed bucket of Calcium Chloride salt, 77 percent pure	put in a sealed 5 gal bucket	
25	6	70 lb bags of concrete sand		
26	64	rapid set repair mortar - sealed 5 gal bucket	CTS	custom packaging
27	16	cans of permanent spray paint, yellow		
28	2	4-ft segment of 1-ft diameter column form		
29	12	All purpose, moisture reactive glue, 18 oz bottle	Gorilla Glue	

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14. ABSTRACT

During the period of March through November 2011, researchers of the U.S. Army Engineer Research and Development Center (ERDC) and the U.S. Air Force Research Laboratory (AFRL) reviewed the use of precast concrete panels for pavement repair applications in the U.S. and around the world. Based on this review, an AFRL designed prototype system was selected for field investigation and was modified to allow for more efficient panel construction and installation for emergency and contingency portland cement concrete (PCC) pavement repairs. Seven precast concrete panels were fabricated for use in an airfield designed PCC test section. Three different prospective repair configurations were evaluated in a simulated airfield using the test panels. Work task items were timed during panel installation to aid in the evaluation of the repair technique effectiveness and to identify areas for optimization. Additional refinements to the system components and installation procedures were made following the field study.

Results of this phase of the investigation were used to develop fabrication and installation procedures and to determine the supplies and equipment required to construct, stockpile, and install panels in the field. A listing of the expendable construction materials and equipment items required to assemble a deployable containerized kit capable of furnishing a minimum of 12 precast panels was generated.

15. SUBJECT TERMS

Precast slabs, Field testing, Expedient Repair	Precast concrete Precast panels Airfield repair	Rapid concrete repair Airfield rigid pavements

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